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MISSOURI RIVER

GAVINS POINT DAM - LEWIS AND CLARK LAKE
NEBRASKA AND SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

UNITED STATES ARMY
CORPS OF ENGINEERS

OMAHA DISTRICT

MISSOURI RIVER
GAVINS POINT DAM - LEWIS AND CLARK LAKE
NEBRASKA AND SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

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MISSOURI RIVER
GAVINS POINT DAM - LEWIS AND CLARK LAKE
NEBRASKA AND SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

PERTINENT DATA

1. EMBANKMENT

Type	Rolled Earth and Chalk Fill
Height Above Stream Bed	74 Feet
Height Above Flood Plain	59 Feet
Length	8,700 Feet
Crest Elevation	1234.0 Feet, m.s.l.
Crest Width	35 Feet
Volume	7,000,000 Cubic Yards
Closure Date	31 July 1955

2. SPILLWAY

Type	Concrete-lined Chute w/Gated Weir
Width	664 Feet
Weir Crest Elevation	1180.0 Feet, m.s.l.
Gates, Type	Radial Tainter
Gates, Number and Size	14-40 Feet by 30 Feet
Elevation, Top of Gates	1210.0 Feet, m.s.l.
Discharge Capacity	584,000 c.f.s.
Maximum Pool Elevation	1221.4 Feet, m.s.l.

3. POWERHOUSE

Length	234 Feet, 8 Inches
Width	187 Feet, 6 Inches
Number of Generating Units	3
Generating Capacity, Each Unit	35,100 Kilowatts
Total Installed Capacity	105,300 Kilowatts
Power on Line	September 1956

RESERVOIR

Drainage Area Above Dam	279,480 Square Miles
Drainage Area, Gavins Point Dam to Fort Randall Dam	16,000 Square Miles
Storage Capacity at Maximum Operating Pool (El. 1210)	540,000 Acre-feet
Exclusive Flood Control (El.1208 to El.1210)	63,000 Acre-feet
Annual Flood Control and Multiple-Purpose (El.1204.5 to El.1208)	102,000 Acre-feet

RESERVOIR

Carry-over Multi-Purpose (El. 1195 to El.1204.8)	220,000 Acre-feet
Dead Storage	155,000 Acre-feet
Length of Pool at El. 1210	25 Miles
Maximum Operating Pool Elevation	1210.0 Feet, m.s.l.
Base, Seasonal Flood Control Pool	1204.5 Feet, m.s.l.

MISSOURI RIVER
GAVINS POINT DAM - LEWIS AND CLARK LAKE
NEBRASKA AND SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

1. INTRODUCTION.

1.1 **Purpose and Scope of Report.** This report provides a summary record of significant design, construction, and operational data on the Gavins Point Dam embankment. It was prepared in accordance with ER 1110-2-1901, "Embankment Criteria and Performance Report," dated 31 December 1981 and is for use by engineers to familiarize themselves with the project, re-evaluate the embankment when needed, and for guidance in designing comparable future projects.

The report presents a general description of the foundation conditions, the type of material and placement methods of the various sections of the embankment, the design considerations on stability and seepage control, instrumentation, significant operational events, and an evaluation of the condition of the embankment. Pertinent drawings, design data, charts, and photos are included. A more detailed description of the foundation conditions is contained in the Construction Foundation Report prepared in 1980.

1.2 **Brief Description and Purpose of Project.** The Gavins Point Dam-Lewis and Clark Lake Project is one of six multi-purpose dam projects on the Missouri River for flood control, irrigation, navigation, power, and recreational purposes. The project is operated and maintained by the U.S. Army Corps of Engineers, Omaha District. The primary purpose of the reservoir is to provide regulation of flows below Fort Randall Dam which is located approximately 69 miles upstream of Gavins Point Dam. Gavins Point Dam is the farthest downstream of the main stem dams and, as shown on the location map on Plate A-1, is located about 4 miles west of Yankton, South Dakota, and about 9 miles north of Crofton, Nebraska. It is 811.1 river miles (1960 adjustment) above the mouth of the Missouri River. The river is the boundary between South Dakota and Nebraska at the damsite.

The project consists of an earth embankment, a gated concrete spillway, and a hydro-electric power generating plant. The embankment is about 75 feet high above the streambed and extends about 8,700 feet from the left abutment to the spillway structure near the right abutment. A plan and typical sections of the project structures are shown on Plate A-2.

1.3 **Authorization of Dam Project**. The Gavins Point Dam and Reservoir project was authorized by the Flood Control Act, approved 22 December 1944 (Public Law 534, 78th Congress, 2nd Session).

1.4 **Design and Construction of Project**. The project was designed by the U. S. Army Corps of Engineers, Omaha District. Members of the Board of Consultants were Dr. Arthur Casagrande, Mr. L. F. Harza, Mr. S. O. Harper, Mr. W. H. McAlpine, and BG Hans Kramer, U.S.A. (Retired).

The embankment was constructed in two stages, Earthwork Stage I (Contract No. DA-25-066-ENG-1603) and Powerhouse Substructure, Spillway, and Earthwork Stage II (Contract No. DA-25-066-ENG-2409). The first stage was awarded in April 1952 to List and Clark Construction Co. and was completed in December 1952. The second stage was awarded in March 1953 to Western, Massman, and Jones, a joint venture of Western Construction Co., Massman Construction Co., and J. A. Jones Construction Co. The river closure was made on 31 July 1955 and the embankment was essentially completed by November 1955. The embankment pressure relief well system was installed by Layne-Western Company in 1955 under Contract No. DA-25-066-ENG-3137. The powerhouse superstructure contract was awarded in September 1954 to Foley Brothers and Donovan Construction Company. All of the contracts were administered by the Corps of Engineers, Omaha District. Field supervision was by personnel of the Gavins Point Area Office which was located in Yankton, South Dakota.

1.5 **Significant Operational Events**. The reservoir is normally maintained at about El. 1208 feet, except in the spring when it is lowered to about El. 1204 feet in preparation for storage of snow melt and storm runoff. Events that have caused some concern and required remedial action are briefly described below.

1.5.1 **Initial Reservoir Filling**. The reservoir began forming with the closure of the embankment on 31 July 1955 and reached the operating level of El. 1204.5 feet, mean sea level, in February 1956. On 27 January 1956, operation of the spillway gates was shifted from gates 1, 2, and 3 to gates 7 and 8. This resulted in an immediate flow increase from the spillway gallery drains from less than 5 gpm to about 15 gpm. The flow continued to increase to a maximum of 170 gpm by 23 March 1956 even as the reservoir was being lowered. The flow was from the central part of the spillway through the 12-inch half-round CMP drain located at the base of the weir beneath the gallery. The reservoir was lowered to about El. 1189 feet by mid-April 1956. The drain flow rate gradually decreased to less than 5 gpm in August 1956. The reservoir was raised to normal operating levels during June and July 1956. Except for a temporary increase to about 18 gpm in 1957, the flow rate has since remained between 5 and 10 gpm. The drain flow increase was attributed to leakage through the spillway contraction joints which opened slightly during the winter.

1.5.2 **Downstream Seepage**. Shortly after the dam was constructed, relief well discharges and foundation seepage downstream of the wells resulted in unsightly surface conditions. This problem was corrected in 1957 by placing a gravel blanket over the wet area and by extending the outlet ends of the well discharge pipes.

In November 1968, a 6-inch diameter seepage boil was noted in the discharge ditch approximately 25 feet from the discharge pipe of relief well No. 36. This condition was not considered critical, but was kept under observation. In April 1969, it was reported as being more active and was corrected in May 1969 by excavating the area and backfilling with 15 cubic yards of clean, well-graded gravel. During this operation, uplift pressures were controlled by pumping the adjacent wells.

1.5.3 **Upstream Berm Erosion**. Riprap was placed in 1973 along the wave erosion scarp which extended over almost the entire length of the upstream chalk berm and which steadily advanced toward the main embankment.

1.6 **Reference Project Publications**. Detailed information on the constructed dam foundation, evaluation of relief wells, project maintenance, and periodic inspections are included in the following Omaha district manual and reports:

- **a.** Construction Foundation Report (1980).
- **b.** Relief Well and Underseepage Report (1963).
- **c.** Operation and Maintenance Manual.
- **d.** Periodic Inspection Reports Nos. 1, 2, 3 and 4, the latest of which is dated May 1981.

2. GEOLOGY.

2.1 **General**. Physiographically the location of Gavins Point Dam is in the central lowland Province. The Missouri River divides the area into the Western Young Drift section in South Dakota and the Dissected Till Plains section in Nebraska. The present course of the Missouri River represents the river's adjustment to flow along the edge of the farthest southward advance of the Wisconsin ice sheet. The characteristic features of the physiographic divisions resulted from glaciation of different periods. In the Dissected Till Plains to the south, pre-glacial features were completely buried as a result of the advance of the Kansan ice sheet. The resulting flat, glacial till plain is submature to mature in its erosion cycle, and has a relief between 100 to 300 feet. A mantle of loess measuring a few feet thick overlies the till. To the north lies the Western Young Drift section. This till has the characteristic distinguishing features of young glacial drift (Wisconsin) such as immature drainage and marginal moraines.

The strata in the general region of the dam site are essentially flat-lying. The exposed foundations are Cretaceous sedimentary beds, which dip gently towards the west. Niobrara chalk is the most prominent Cretaceous formation, varying in thickness from 165 to 185 feet. It commonly forms prominent cliffs along the river for some 190 miles from the Gavins Point Dam site near Yankton, South Dakota, to the vicinity of Big Bend Dam near Fort Thompson, South Dakota. Pierre shale overlies Niobrara chalk, rising above

the chalk cliffs as grass covered slopes. At the dam site the Niobrara chalk is overlain by Pierre shale only on the high knolls above and beyond the left abutment. Pleistocene glacial deposits cover the surface on both sides of the river. Most of the large valleys of this region contain alluvial deposits which are regarded as recent or post glacial age.

The width of the Missouri River valley at the dam site is approximately 8,000 feet. Prior to construction of the dam, the river course was divided into two main channels and several small chutes by an island formed of alluvial material. This island was approximately 9,000 feet long and 6,000 feet wide and was built up to a maximum elevation of 1178 feet, about 9 feet above the normal river stage. The main channel was approximately 700 feet wide and was located along the left side of the valley. The channel along the right side of the valley was relatively narrow and averaged about 200 feet in width. At the right abutment, the ground rises abruptly from the base of the bluff to about El. 1270 feet, from where it gradually slopes to an elevation of about 1300 feet. Niobrara chalk is exposed along the bluff. The bank line on the left side of the river is approximately at El. 1190 feet. From the bank line to a distance of about 2,000 feet, the ground surface gradually rises to El. 1300 feet, then becomes much steeper.

2.2 **Subsurface Explorations**. From August 1948 to the beginning of the Earthwork Stage I construction in April 1952, more than 250 6-inch diameter holes were drilled at and in the vicinity of the dam site. Twenty-two seismic lines were "shot" and 29 auger holes were drilled to supplement the data from the core borings to determine the configuration of the top of bedrock. About 50 additional holes were drilled prior to awarding of the Earthwork Stage II contract in March 1953. The locations of about 230 of the borings drilled up to the Stage II contract are shown on Plates A-8 and A-9 and the boring logs are presented on Plates A-10 through A-17. As shown on Plates A-8 and A-9, a large number of the holes were drilled slightly downstream of the dam axis in the originally proposed dam location. The present location was selected after borings revealed a wide bedrock shelf for the spillway structure foundation about 600 feet upstream of the original site. Prior to construction of the project, the bedrock was mapped from data obtained from

borings and from observation of the exposed natural bluffs. During construction of Earthwork Stages I and II, the bedrock conditions were evaluated by drilling additional holes and by observing excavations as they were being made for the powerhouse and spillway structures. Water pressure tests in drill holes were also performed during construction to evaluate the tightness of the bedrock. A geologic profile along the centerline of the dam is shown on Plate A-6. Details of the foundation investigation, bedrock mapping, pressure testing, and grouting of the bedrock in the vicinity of the powerhouse and spillway are presented in the referenced "Construction Foundation Report (1980)."

2.3 **Bedrock.**

2.3.1 **General.** Niobrara chalk and Carlile shale of the Colorado group, Middle Cretaceous in age, comprise the bedrock at the dam site. Niobrara chalk overlies Carlile shale and the contact between these two was encountered in many bore holes. Niobrara chalk is exposed in most of the bluffs on both sides of the river, whereas Carlile shale occupies the valley bottom. The elevation of this contact varies between 1120 and 1150 feet, but is mostly around 1140 feet along the alinement of the dam axis and the surrounding areas. The entire reservoir area in general is situated in Niobrara chalk and Carlile shale. Niobrara chalk is quite impervious and does not tend to develop any solution channels. The seepage out of the reservoir was expected to be at a minimum.

2.3.2 **Physical Characteristics and Strength of Bedrock.** The overburden and bedrock materials found at Gavins Point Dam were very similar to those at Fort Randall Dam which had been extensively tested. Consequently, the testing program for the Gavins Point project was primarily for identification and correlation purposes. More emphasis was given to the Carlile shale formation as it was the weaker bedrock material and was to be the foundation for the concrete structures.

2.3.2.1 **Niobrara Chalk.** Niobrara chalk generally occurs in beds ranging from 6 inches to 3 feet in thickness. At the damsite, there

is a well developed box work of gypsum-filled joint planes cutting the formation in several directions. Some minor faults were observed during excavations. In general, the chalk is homogeneous and blocky. There are several beds of buff, gray and purple colored bentonite interbedded in the chalk. None of the bentonite beds observed exceeded two inches in thickness. The bentonite beds are most numerous at the base and again near the top of the formation. There are no records of any solution passages or water-bearing cavities ever having been found in the chalk in this region. The chalk has 40 percent porosity, but low permeability due to its fine grained nature. Its dry weight varies from about 80 to 115 pounds per cubic foot. No solution action was observed, but considerable weathering occurs in winter and spring because of cyclical freezing and thawing. Unconfined compression tests were made on 32 chalk samples from the Gavins Point dam site. The results showed an average breaking strength of 750 psi with the load applied normal to the bedding plane. Three tests showed strength below 500 psi and three tests were above 1200 psi. Stress applied lateral to the bedding plane showed strengths approximately 60 percent of those obtained from the normally loaded tests.

2.3.2.2 **Carlile Shale**. There are three members in the Carlile shale. The topmost is Codell sandstone, followed by the Blue Hill shale below, with Fairport shale at the bottom. The Codell sandstone phase of the Carlile shale has not been encountered in the vicinity of the dam site. The Blue Hill member is approximately 120 feet thick and consists of dark gray to black argillaceous shale. The Fairport shale member underlies the Blue Hill member and is about 60 feet thick. This member is a black calcareous shale, interspersed with thin limy layers and microfossils. Carlile shale is very fine grained and generally waxy in appearance and feel. Except for a 5- to 10-foot thick well-cemented sandy phase at the top of the formation, Carlile shale is a compaction type sediment which has undergone little or no cementation and is free from bentonite beds and has a uniform texture and firmness throughout. The shale is thin bedded in the lower portions, where it is argillaceous and calcareous. It is impermeable and groundwater has little effect on it. The sandy phase in the upper portions is well-cemented and indicates little permeability. This sandy stratum has a

bearing strength greater than that of average Carlile shale and is chemically insoluble by ground water. This sandy zone is referred to as the "sandy phase" in the design reports and drawings. The argillaceous Blue Hill shale member is fine-grained and is referred to as the "clay phase" of the Carlile formation. Atterberg tests on the clay phase of the Carlile indicate that the material generally has a liquid limit of about 54 and a plasticity index of about 32. Tests on 146 6-inch core samples of the clay phase material showed an average in-place density of 137 pounds per cubic foot, with a dry density of 118 pounds per cubic foot and a moisture content of 16 percent. Unconfined compression tests on 133 clay phase samples showed an average breaking strength of 240 pounds per square inch. Results of direct shear and triaxial shear tests on the clay phase material were used to select the foundation design shear strength values of tan $\emptyset = 0.3$ and cohesion = 0.2 tons per square foot.

2.4 **Overburden.** The overburden materials are composed of glacial deposits on both sides of the valley and primarily alluvial deposits in the river valley. The glacial deposits are heterogeneous mixtures of silt, clay, sand, and gravel with numerous boulders dispersed in them. Small lenses of sand and gravel are found in the general mixture of glacial drift on the right abutment. The thickness of these glacial deposits varies between 0 and 20 feet in the vicinity of the dam site. The river valley deposits are chiefly composed of fine to coarse sand with some layers containing gravel, silt, and clay. Information from borings indicate that downward cutting and later filling extends to depths of about 150 feet below river level. A geologic profile along the axis of the dam is shown on Plate A-6. A profile developed from borings along the toe of the embankment is shown on Plate A-7. The embankment is founded on the valley alluvium overlying the Carlile shale bedrock. The depth of the alluvial deposit varies from about 35 feet near the spillway structure to about 150 feet beneath the central portion of the embankment. The overburden beneath the north end of the embankment generally varies from about 45 to 60 feet in the embankment closure area to about 90 feet in the left bank area. The upper 20 to 40 feet of the left bank overburden is composed of clays. Except for the exposed alluvial sands in the river channels, the valley alluvium is covered with a thin relatively

impervious blanket of silts, sandy silts, and clays. A design shear strength of Tan $\emptyset$ = 0.6, cohesion = 0 was assumed for the alluvial material (see Plate A-25).

3. **CONSTRUCTION STAGES**. The embankment was constructed in two stages under separate contracts. These are briefly summarized below.

3.1 **Earthwork Stage I**. The first earthwork contract included partial construction of the embankment, primarily the upstream impervious blanket, from approximately Station 27+65 near the spillway to about Station 70+65 near the north side of the island. This included filling of a 200-foot wide river chute just north of the spillway and a wider 1,100-foot chute that cut through the island. Earthwork Stage I also involved initial excavation in the spillway to about El. 1170 and in the powerhouse area to about El. 1210. Chalk and overburden material from the structure excavation and overburden material from the structure excavation and from the borrow area south of the powerhouse were used as fill for the embankment. Approximately 1,080,000 cubic yards of chalk and 850,000 cubic yards of overburden were excavated and placed in the embankment section. Profiles and sections of the Stage I embankment are shown and noted as "placed by prior contract" on the Stage II drawings, Plates A-18 through A-21. Photos No. 7 and 8 show the dike constructed across the upstream portion of the central river chutes.

3.2 **Earthwork Stage II**. Earthwork Stage II was constructed under the same contract as the construction of the powerhouse substructure and spillway. Work under this contract included diversion of the river flow, closure of the embankment, and completion of the embankment in addition to construction of the spillway structure and the powerhouse substructure. Embankment materials included chalk and shale excavated from the structure area, overburden soils from the diversion channel and from the right abutment and left bank borrow areas, dredged alluvium from downstream of the dam, and graded pervious filter material from the river alluvium and from other sources. Also included in this contract was the construction of a training dike for the spillway discharge channel. Approximately 3,200,000 cubic yards of chalk

and shale, about 2,800,000 cubic yards of valley alluvium, and 1,200,000 cubic yards of left and right bank overburden were excavated under this contract. These were mostly used for construction of the embankment. Pertinent drawings showing the Earthwork Stage II construction work are presented on Plates A-18 through A-22. Photos No. 10 through No. 22 were taken during this phase of construction.

4. **FOUNDATION PREPARATION**. All areas upon which embankment material were placed, plus at least a 10-foot contiguous strip, were cleared of all brush, trees, structures, trash, debris, and other unsuitable foundation material. Roots larger than 1-1/2 inches in diameter were removed to a minimum depth of 3 feet below the ground surface. Thin surface layers containing sod, humus, and other undesirable material were stripped and wasted. Depressions were filled with either compacted impervious fill or compacted chalk material. Prior to placement of embankment, the foundation was loosened to a depth of 12 inches by scarifying, plowing, or harrowing, cleared of loosened roots and debris, then compacted as for impervious fill. In river chute areas, however, the flows in the chutes were first blocked off with chalkfill dikes constructed across the chutes at the upstream edge of the embankment. Random fill consisting of pervious alluvial material was then placed in the chute to about El. 1170, slightly above the normal river stage.

5. **EMBANKMENT SECTION**. The embankment has a maximum height of about 75 feet above the river bed and an average height of about 60 feet across the flood plain. The crest of the embankment is 35 feet wide and is at El. 1234.0 feet, mean sea level. The upstream face is sloped 1V on 4H from the crest to El. 1217, then 1V on 15H to El. 1203, and 1V on 3H to the top of the impervious blanket. The downstream face is on a 1V on 2.5H slope from the crest to El. 1215, then on 1V on 3H to the top of the chalk berm which supports a toe road across the full length of the valley embankment. The elevation of the toe road varies from about El. 1180 to El. 1185 feet. A typical section of the embankment is shown on Plate A-2 and sections along the embankment are shown on Plates A-4 and A-5. As-built embankment plan, profile, and

sections are on Plates A-18 through A-22. The embankment is composed predominantly of impervious earth fill and chalk fill. The design of the embankment was based on the use of the large quantity of chalk from the powerhouse and spillway excavations. Random fill in the lower elevations and filter material in the inclined drain section in the downstream side of the impervious core are other types of fill in the embankment.

Seepage control within the embankment is provided by a conventional central impervious core. Continuity of the impervious section is provided to a distance of about 600 feet upstream of the dam axis by a horizontal impervious blanket immediately upstream of the core and a relatively impervious compacted chalk fill blanket. The blanket extends across the valley and ends abruptly at the left bank of the river where a 20 to 40-foot thick surface layer of impervious silt and clay provides a natural seepage control blanket.

Underseepage control is provided primarily by a system of 48 relief wells along the downstream toe of the embankment. Flows from the wells are discharged into Lake Yankton, a shallow-lake that was formed by construction of the spillway discharge channel training dike and of a causeway embankment across the natural river channel approximately 1 mile downstream of the dam.

6. **MATERIALS AND MATERIALS PLACEMENT**. The embankment was constructed of material obtained from required excavations and from left and right bank borrow areas. The materials were placed in a manner to assure a stable structure which consisted of an impervious core and impervious upstream blanket, compacted chalk outer sections, and a relatively flat upstream uncompacted chalk berm. Maximum use was made of the large quantities of chalk material excavated from the spillway and powerhouse areas and of impervious and pervious overburden material from these areas and from the diversion channel. Impervious material for the core and upstream blanket was obtained from the borrow areas. Placement and compaction methods were similar to those used at Fort Randall Dam which was constructed just prior to Gavins Point Dam. Records on field compaction tests and embankment construction control were sent to the federal records center in Kansas City, Missouri, and were subsequently destroyed. However, laboratory test records on 20 undisturbed box

samples of the embankment material taken for record purposes during construction are available. Tests on these samples included moisture content, density, permeability, consolidation, direct shear, unconfined compression, and one consolidated-undrained triaxial shear test. Also available are classification and moisture content data on samples from five piezometer holes that were drilled during the Stage II earthwork construction.

6.1 **Uncompacted Random Fill**. Uncompacted random fill was used to fill the river chutes to about El. 1170 beneath the upstream embankment section from the upstream chalkfill dike to about the embankment centerline. It consisted of relatively pervious alluvial sands from the spillway, diversion channel, and discharge channel excavations and also pervious material from the borrow areas. In Earthwork Stage I, the material was end-dumped in the south and central river chutes. In Earthwork Stage II, it was placed in the upstream part of the embankment closure section by means of hydraulic dredging after initial closure was accomplished by construction of a chalkfill cofferdam across the main river channel.

6.2 **Compacted Random Fill**. Compacted random fill consisted of the same type of materials as uncompacted random fill. It was placed beneath the impervious core, upstream compacted chalk blanket, impervious upstream blanket, and downstream compacted chalk fill. In Earthwork Stage I, this material was placed beneath the upstream embankment section from El. 1170 to about El. 1177, sandwiched between the underlying uncompacted random fill or prepared foundation and the overlying compacted chalk and impervious fill blankets. In Earthwork Stage II, it was placed from the original ground line to El. 1177 beneath the upstream embankment, impervious core, and filter sections and to El. 1185 beneath the downstream compacted chalk section. The material above water was placed in 18-inch loose lifts and was compacted by at least three complete passes of a 50-ton rubber-tired roller (Photo No. 9). Underwater placement was by hydraulic dredging of pervious alluvial material. (Photo No. 18).

Two undisturbed box samples were taken of the compacted random fill. One was from El 1177.5, Station 80+24, and Range 5137 and the other was from

El 1180.5, Station 80+80, and Range 5122. These samples were classified as SP sands with dry densities of about 91 and 101 pcf (pounds per cubic foot). A permeability test on the denser sample showed a permeability coefficient of 4.5×10^{-3} cm/sec. Samples obtained from a piezometer hole located at Station 75+00, Range 4980 indicated that the materials in the random fill section at that location varied from clayey sands to lean clays.

6.3 Impervious Fill. Impervious fill was placed in the central core and in the connecting upstream impervious blanket which extended to a distance of about 300 feet from the dam axis. The impervious blanket is at least 8 feet thick at the upstream end and becomes much thicker at the central core. Impervious material consisted of clays, silty clays, and clayey silts from the right abutment and left bank borrow areas. No Atterberg limits were specified. Moisture content was specified to be from optimum to wet of optimum to insure a flexible impervious core section to minimize cracking that could occur from differential settlement. The material was placed in 12-inch thick loose lifts and compacted by at least three complete passes of a 50-ton rubber-tired roller.

Tests on four undisturbed samples from the core section and two from the upstream impervious section revealed that the materials in both sections were generally of similar types. Materials consisted predominately of sandy clays (CL), but also included fat clays (CH). Sand content in the clays ranged from about 25 to 40 percent. Atterberg limits of the CL clays varied from LL=31, PI=13 to LL=41, PI-25. Dry densities (DD) and moisture contents (w) varied from DD=107 pcf, w=16.7 percent to DD=114 pcf, w=15.2 percent. The two CH clay samples had the following properties: LL=52, PI=31, DD=82 pcf, and w=20.3 percent for one sample and LL=67, PI=42, DD=94 pcf, and w=24 percent for the other. Falling head permeability tests indicated a permeability coefficient of approximately 1.0×10^{-7} cm/sec. Seven unconfined compression tests showed breaking strengths varying from 1.17 to 2.87 tons per square foot (TSF) and an average strength of about 2.2 TSF. Results of two direct shear tests are tabulated on Plate A-26.

Samples taken from three piezometer holes located at Station 50+00 (Range 4980), Station 75+00 (Range 4980), and Station 75+00 (Range 4850) were predominately lean and sandy clays. In general, the moisture contents were at or above optimum.

6.4 **Compacted Chalk Fill**. Compacted chalk fill was placed in upstream and downstream sections of the main embankment and in the upstream compacted chalk blanket. The chalk blanket is a 300-foot extension of the upstream impervious blanket and varies in thickness from 8 feet at the impervious blanket to 5 feet at the upstream end. The material was composed of chalk and shale excavated from the spillway and powerhouse areas. Properties of the compacted chalk fill were determined from a chalk test embankment previously constructed at Fort Randall Dam. Based on the test fill, the compacted chalk fill for Gavins Point Dam was placed and compacted, as follows: The chalk was dumped and spread in 12-inch loose layers and further broken down by at least four complete passes of a spike-tooth roller. Moisture was added, as needed, during the spike-tooth rolling. The roller was basically a tamping roller with the tamping feet modified into removable 10-1/2-inch pointed teeth. Final compaction of the material was obtained by at least six complete passes of a tamper roller or by three complete passes of a rubber-tired roller. No particular difference in operation or results was noted between the two compaction methods, although the contractor generally preferred the rubber-tired roller because of the fewer required passes. In the downstream compacted chalk fill section, the finer graded material was placed adjacent to the filter section.

Tests on 12 undisturbed box samples of the compacted chalk indicated that the material is generally of CH and CL clays containing approximately 5 to 20 percent sands. Atterberg limits ranged as follows: LL=54, PI=21 for the CH clays to LL=33, PI=11 for the CL clays. Laboratory permeability tests indicated a permeability coefficient of 2.6×10^{-4} cm/sec for a ML silt sample and 2.3×10^{-6} cm/sec for a CH clay material. The average dry density of the compacted chalk was about 86 pounds per cubic foot which was higher than that obtained in the Fort Randall test embankment. The average moisture content was 28 percent which was about 6 percent lower than the moisture content in

the test embankment. Eight unconfined compression tests showed a wide range in breaking strength, from 0.8 TSF to 46.6 TSF. The six lowest results ranged from 0.8 to 4.0 TSF and averaged 2.5 TSF. The only consolidated-undrained (R) triaxial shear test conducted on the record samples was made using three specimens prepared from box samples taken from different locations within the compacted chalk section. The specimens, therefore, were not uniform and were classified as CH, ML, and CL soil types with dry densities of about 95, 75, and 85 pcf, respectively. The resulting strength envelope indicated shear strength parameters of Tan Ø=0.28 and cohesion =1.02 TSF. Because of the limited test data, "R" strength parameter of Tan Ø=0.35 and C=0.35 TSF were conservatively selected for the stability reevaluation d' ussed below under "Embankment Stability." Results of four direct shear tests on the compacted chalk fill are tabulated on Plate A-26.

6.5 Uncompacted Chalk Fill. Uncompacted chalk fill consisted of chalk and shale from the powerhouse and spillway excavations. The material formed the outer upstream embankment section consisting of the thick 1V on 15H berm section and the thinner 1V on 4H upper main embankment section. The specifications allowed the material to be placed in horizontal layers 5 feet or less in thickness, and be compacted only by the passage of the hauling and spreading equipment. A large part of the uncompacted chalk fill section was placed by this method; however, the Earthwork Stage II contractor requested and was allowed to place a section by hydraulic filling. A 30-inch cutter-head, diesel-electric dredge was used to excavate the firm chalk from the discharge channel downstream of the powerhouse and spillway. The chalk was pumped through a 30-inch line approximately 4,000 to 6,000 feet in length and deposited in the chalk section. The exact location of the hydraulic fill is not known, but is estimated to be over a portion of the island from about Station 60+00 to Station 75+00. The dredged fill was placed over 3 feet of conventionally placed uncompacted chalk and was capped with a 3-foot cover of firm gray chalk fill. Placement by this method was considered very satisfactory. The relatively flat upstream berm section was assumed to be sufficiently resistant to wave action and was, therefore, constructed without any type of protective cover. However, wave erosion started to occur as soon as

the pool reached the level of the berm. The progress of wave erosion and subsequent placement of riprap are described below under "Wave Protection."

6.6 **Filter Section**. The filter section is 8 feet wide and lies between the central core and the downstream compacted chalk section. The top of the filter section is at El. 1210 and the bottom is at El. 1177 and in contact with the underlying alluvial foundation or compacted relatively pervious random fill. Drainage of the filter section is through the underlying material and the pervious random fill that was placed immediately downstream of the filter section to EL. 1185. The filter material was required to meet the following gradation:

Sieve Size, U.S. Standard Square Mesh	Percent by Weight Passing
3/4 Inch	100
3/8 Inch	95-100
No. 4	90-100
No. 10	80-100
No. 16	70-95
No. 40	35-80
No. 100	0-45
No. 200	0-20

The material was placed in 18-inch loose lifts and compacted by three passes of a rubber-tired roller.

7. **WAVE PROTECTION**. The flat 1V on 15H upstream chalk berm was assumed to provide adequate resistance against wave action. However, berm erosion started when the reservoir was initially filled in 1956. Erosion progressed towards the main embankment at a rate of about 5 to 10 feet per year and by 1973 had advanced about 150 feet to about range 4850, approximately 60 feet from the toe of the 1V on 4H upper embankment slope.

In 1973, stone protection was placed along the entire length of the erosion scarp, as indicated on Plates A-23 and A-24. Except for a 2,500-foot long test section, the stone protection consisted of dumped field boulders

placed at a rate of 4.0 tons per lineal foot of scarp. The test section included layered riprap on 1V on 2H and 1V on 3H slopes, gabions, dumped quarried boulders, and dumped field boulders. The layered 1V on 2H riprap showed signs of excessive stone displacement within two years after placement and was almost completely washed out by 1979 at which time the 500-foot section was protected with dumped quarried stone placed at a rate of 1.25 tons per lineal foot. Included in this repair work was the placement of quarried stones to repair several small isolated severely damaged areas along the dumped field boulder protection.

The upstream chalk fill portion of the embankment adjacent to the north spillway wall is protected against waves by a 3-foot thick layer of riprap over 1 foot of spalls and 1 foot of filter material. The riprap is well-graded and varies in stone size from a minimum of 5 inches to a maximum of about 3 feet. A plan and typical sections of the riprapped area are shown on Plate A-21. The outer riprapped slope was damaged several times by waves during periods of high winds. Repairs were made by the addition of rock by project forces.

The 1V on 4H upstream slope and the entire downstream slope of the embankment have 9 inches of topsoil and are grassed for protection against surface erosion.

8. **DIVERSION AND CLOSURE**. During the embankment closure operations, the Missouri River was diverted through the powerhouse draft tubes and over the spillway weir. Prior to the closure, the upstream dike was opened to allow the river to flow through the diversion channel to the powerhouse and spillway structures. Fill placement for the closure section was started in June 1955 and initial closure was accomplished at about 4:00 a.m. on 31 July 1955. Closure was made by constructing a chalk diversion dam across the river along the upstream end of the impervious blanket. Photos No. 14 through No. 20 were taken during construction of the closure section. Plans, sections, and details of the closure section are shown on Plate A-22.

9. **SEEPAGE CONTROL.** Seepage through the embankment is controlled primarily by the impervious core, impervious upstream blanket, and the pervious filter section on the downstream side of the core, and the downstream random pervious fill. Underseepage control is provided by the upstream impervious blanket, chalk fill, and pressure relief wells along the downstream toe of the embankment.

9.1 **Relief Wells.** The embankment section was designed by assuming that uplift pressures, especially at the downstream toe of the embankment, would be controlled by pressure relief wells. Because of insufficient boring data, the design of the relief wells was deferred and installation of the wells was not included in the embankment earthwork contract. The necessary borings and tests were made during Earthwork Stage II and the well design was completed in 1954. Relief wells No. 1 through No. 41 were installed during May and June 1955 prior to the embankment closure in July 1955. The remaining wells, No. 42 through No. 48, were installed in August and September 1955 after the closure was made. The wells are spaced from 100 feet apart in the closure area to 250 feet apart at the south end of the embankment, but the majority of the wells are at spacings of 130 and 140 feet. The wells are fully penetrating at the north and south ends of the system where the valley alluvium is relatively shallow. In the central reach where the bedrock is at a much lower depth, the wells penetrate to 75 percent of the thickness of the alluvium. The riser and screen sections of the relief wells consist of 8-inch inside diameter wire-wrapped wood stave pipes. Each well includes a 36-inch diameter corrugated metal pipe (CMP) well pit and an 8-inch diameter CMP discharge pipe through the toe road berm. Seventeen well point type piezometers were installed between selected relief wells to monitor the uplift pressures in the line of wells. Plate A-3 shows the locations of the relief wells and piezometers. Relief well and piezometer details and tabulated location and elevation data are presented on Plate A-6. On Plate A-7 is a profile of the relief well system. Relief well design computations are presented on Plates A-29 and A-30 and the design of the gravel pack gradation is on Plate A-31. A plot of total well flow is shown on Plate A-32 and typical plots of individual well flows are presented on Plate A-33.

An evaluation of the performance and effectiveness of the relief well system was made in 1963 and the results are presented in the referenced report, "Relief Well and Underseepage Report," dated April 1963. The evaluation showed that the wells were very effective in controlling underseepage. Monitoring of the relief wells and piezometers indicates that the wells continue to provide the necessary underseepage control. All of the wells were pump tested in 1980 and were determined to be generally in good condition.

9.2 **Seepage Control at Spillway Contact.** To provide adequate seepage control where the embankment abuts the north spillway wall, the valley alluvium at that location was excavated on a 1V on 2H slope above the top of the wall footing approximately 20 feet below the natural ground surface. Impervious core and blanket materials were then placed and compacted against the structure. In addition, a thick impervious blanket was placed directly beneath the approach channel riprap and tied to the top of the shale bedrock by a cut-off trench. This impervious section was constructed to lengthen the seepage path along the wall and extends from the spillway wall to the upstream end of the chalk blanket. Details of the embankment construction at the spillway wall are presented on Plate A-21.

9.3 **Seepage Control at Left Bank.** The embankment foundation on the left (north) bank of the main river channel consists of approximately 20 to 40 feet of clays which provide an adequate impervious blanket against underseepage pressures. A 5-foot thick random pervious fill blanket was placed beneath the embankment section downstream of the impervious core to drain seepage that occurs through or under the embankment. Details of this blanket are shown on Plate A-19.

10. **EMBANKMENT STABILITY.** The stability of the embankment was analyzed during the project design stage in 1951 and was re-evaluated in 1976 using then current criteria. In both instances, the embankment at about Sta. 30+00 just north of the spillway was selected for analysis. This location was selected because the weaker, weathered clay phase of the Carlile shale is at a much

shallower depth, about 35 feet, than in the central valley section where the higher strength alluvium extends to a depth of over 90 feet.

10.1 **Original Stability Analysis.** The design stability analysis on the critical embankment section is presented on Plate A-25. Assumed water and seepage conditions for the steady seepage condition and adopted embankment and foundation shear strengths are indicated on the plate. The analysis showed a minimum factor of safety of 1.59.

10.2 **Stability Reevaluation.** A reevaluation of the embankment stability using current criteria was made in 1976 at the request of members of the Office of the Chief of Engineers who participated in the 1971 periodic inspection. The report was submitted as Appendix Q of Periodic Inspection Report No. 3, dated May 1976. The steady seepage, sudden drawdown, and partial pool cases were analyzed by the wedge method of analysis described in EM 1110-2-1902, 1 April 1970, "Stability of Earth and Rockfill Dams." All analyses were performed by computer using a WES program, 741-G9RP-107, "Slope Stability Wedge Method," and critical cases were checked by hand computations. As expected, the analysis of the flat upstream slope revealed high safety factors of 2.7 for the sudden drawdown case and 2.0 and 3.2, with and without earthquake, respectively for the partial pool case. The critical steady seepage case showed safety factors of 1.49 and 1.42 for the spillway pool and surcharge pool conditions, respectively. With application of a seismic coefficient of 0.1, the analysis showed a safety factor of 1.05 for the steady seepage, spillway pool condition.

Initial reevaluation using the original design shear strengths indicated factors of safety in the range of 1.1 to 1.2 for the steady seepage case. It was recognized, however, that all of the original design strengths were based on the lowest strength achieved for the testing program. Consequently, for the reevaluation, shear strengths were selected as follows:

a. The shear strengths of the embankment chalk and random fill materials were based on the results of shear tests made on undisturbed record box samples taken during construction of the dam.

b. The strengths of the embankment impervious fill, foundation alluvium, and shale bedrock were adjusted from the lowest strengths to a value selected such that two-thirds of the test values exceeded the adopted value. Although the alluvium is primarily sandy material, it contains lenses of silt and clay; therefore, strengths applicable to these impervious types of materials were used. The original design assumed that the alluvium was composed of a sandy type material.

Material properties and adopted shear strengths are presented on Plates A-26 and A-27 and the manual computation of the critical wedge stability analysis is shown on Plate A-28.

11. **SETTLEMENT.** Overbuild of the embankment crest was not considered necessary as most of the settlement of the pervious foundation was expected to occur during construction of the embankment. Based on the settlement recorded at Fort Randall Dam, the maximum settlement of the Gavins Point embankment was predicted at about 1.5 feet. Eighty percent of the settlement was expected to occur during construction and the remainder in about 6 months after construction. The predicted settlement was confirmed by settlement gage readings during and after construction of the project. The readings indicated that settlement stabilized after a maximum settlement of about 1.6 feet in a span of about 25 years. Approximately 65 percent of the settlement occurred by the time the full height of the embankment was constructed and an additional 15 percent occurred by the end of the project construction. The left bank foundation has a thicker clay deposit than the valley alluvium, but has slightly less embankment load. It has settled a maximum of about 1.75 feet, 75 percent of which occurred during construction of the project. Typical settlement plots are shown on Plate A-38. Settlement gages are described below under "Instrumentation."

12. **INSTRUMENTATION.** Instrumentation of the Gavins Point embankment consists of settlement gages, crest and slope movement markers, embankment piezometers, relief well piezometers, and strong motion accelerographs.

12.1 **Settlement Gages.** During construction of the embankment, nine settlement gages were installed, three each at locations 27+30, 56+00, and 99+50, as indicated on Plate A-37. Seven of the gages remain functional. Gages A and G on the upstream berm were destroyed by ice action and wave erosion in 1957. The gages consist of 6-foot diameter, 1/2-inch thick steel plates installed on the prepared foundation immediately beneath the embankment. At each location, a 2-1/2-inch diameter steel pipe was connected to the plate and was extended through the fill as construction of the embankment progressed. The pipe was extended inside a 4-inch diameter protective steel pipe from about 6 feet above the plate and both pipes were capped above the surface of the embankment. Readings have been taken at regular intervals and typical plots are shown on Plate A-38 for Gages A, B, and C which are all located at Station 56+00.

12.2 **Crest and Slope Movement Markers.** The locations of 13 crest and slope movement markers are shown on Plate A-37. Initially, the markers consisted of concrete monuments extending approximately 5 feet into the embankment. Concrete monument reference points were also set in the abutments and downstream of the embankment on the left bank of the river. All of the markers were replaced with deeper "frost free" markers either in 1972 or in 1979. Markers 48B2, 86A2, 86B2, and 94B2 were installed in 1972; each consists of a 10-foot long 2-inch diameter pipe centered in a 6-inch or 8-inch diameter casing. The lower 5 feet of pipe is uncased and is embedded in concrete. The space between the pipe and the casing is filled with vermiculite. Markers 30A, 30B, 40A, 48A, 60A, 70A, 70B, 94A, and 100A were installed in 1979 and each consists of a 1-inch diameter steel rod 10 feet long driven into the ground through a 4-inch diameter, 4-foot deep cased auger hole. The top of the casing extends about 1.5 feet above the ground and is provided with a removable cap. Since July 1979, movement readings have been taken by geodolite survey instead of the former "offset from line of sight" survey. Typical plots of movement marker readings are shown on Plates A-39 and A-40.

12.3 **Embankment and Relief Well Piezometers.** Embankment piezometers were installed in four lines, A through D, normal to the axis of the embankment. Their purpose is to monitor uplift pressures that develop in the alluvial foundation beneath the embankment. Relief well piezometers were installed at select locations between some of the relief wells to monitor the effectiveness of the relief well system along the downstream toe of the embankment. All piezometers are well-point types similar to that shown on Plate A-6. Locations of all of the piezometers are shown on Plate A-3 and are tabulated on Plate A-6.

Six of the original 22 embankment piezometers were destroyed through wave erosion of the upsteam berm. At present, there are 18 embankment piezometers that measure underseepage pressures. They include piezometer A'5.165 which was installed in 1977 near the "A" line and piezometer 79-1 which was installed in 1979 in the closure section. Two other piezometers, 79-2 and 79-3, were also placed in the closure section. However, their tips were set in the chalk fill and filter drain sections, respectively, and at a shallow depth at about the level of the normal pool. These two piezometers, therefore, are not for monitoring underseepage, but are primarily for the detection of seepage through the upper embankment section. Typical plots of embankment piezometer readings are shown on Plate A-34. On Plates A-35 and A-36 are plots of piezometer readings on the embankment sections at lines A, B, C, and D. These show the effective upstream seepage resistance provided by the embankment section and the natural impervious blanket and also show the underseepage gradient through the embankment.

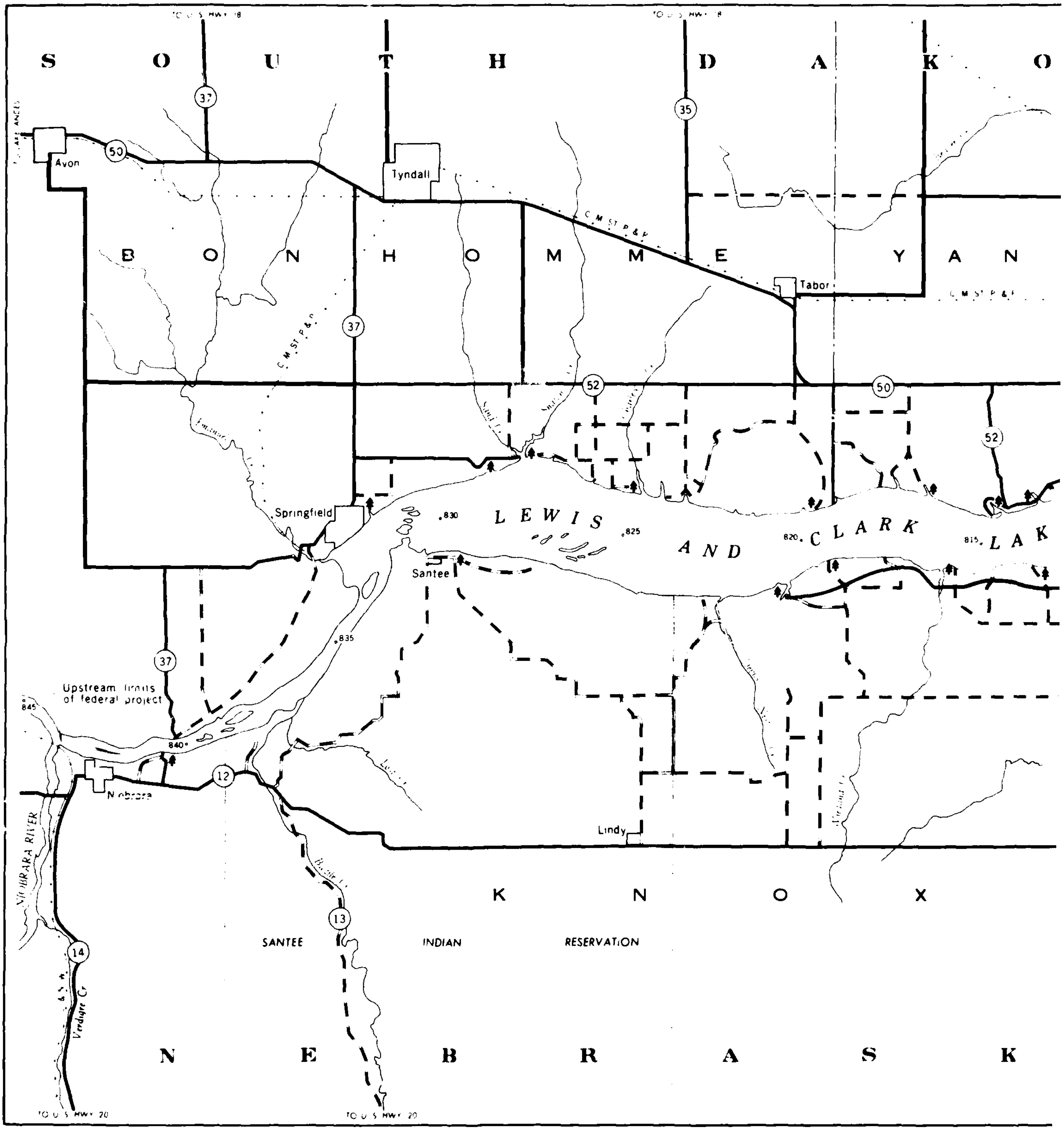
Seventeen piezometers were installed between selected relief wells. These are shown in plan on Plate A-3 and in profile on Plate A-7. A tabulation of the well locations and elevations is presented on Plate A-6. Typical plots of the piezometer readings are shown on Plate A-33. They indicate relatively constant uplift pressures which are safely below those assumed in the design of the relief wells. The relief well system is described above under "Seepage Control."

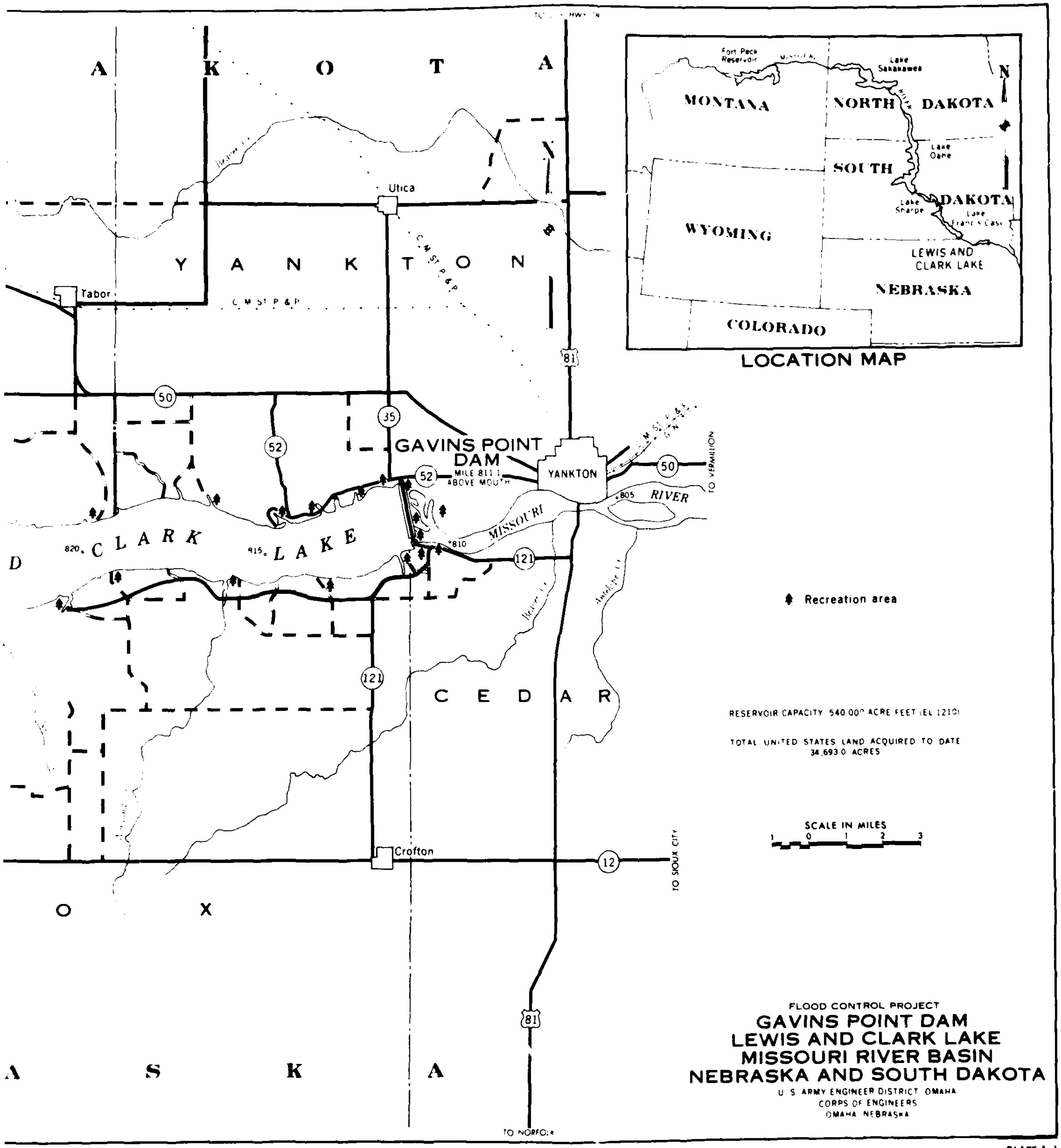
12.4 **Strong Motion Accelerographs.** Gavins Point Dam is located in Zone 1, a low seismic activity region outlined in the seismic probability map, Figure 6, EM 1110-2-1902. Three Kinemetrics SMA-1 strong-motion accelerographs were installed at the project in 1976. One instrument is located on the crest of the embankment at about Station 55+00. Another is also at Station 55+00, but is located in the Cottonwood recreation area about 500 feet downstream of the embankment crest. The third recorder is located in the spillway gallery. All of the instruments were installed and are maintained by the U.S. Geological Survey.

13. **OPERATIONS AND INSPECTIONS.** The Gavins Point project is operated and maintained by the U.S. Army Corps of Engineers, Omaha District. The project office is located in the powerhouse complex and is staffed by permanently assigned operations and maintenance personnel. Annual inspections of the project are conducted by personnel of the district office and periodic in-depth inspections are made jointly by members of the Omaha District and the Missouri River Division, and occasionally the Office of the Chief of Engineers. These inspections are made to assure the structural and operational soundness of this multi-purpose dam project. Periodic inspections are made in accordance with the requirements of ER 1110-2-100 and to date, such inspections have been successfully conducted in 1967, 1971, 1976 and 1981. Results of the inspections are included in the referenced periodic inspection reports.

14. **EVALUATION.** The Gavins Point embankment is in good structural condition. In over 25 years of operation, no serious stability problems have occurred. Instrumentation readings indicate that settlement of the embankment foundation has stabilized, that no unusual embankment deformations are occurring, and that the relief wells continue to provide an effective system of underseepage control. In addition, daily surveillance by project personnel and annual and periodic inspections by members of the District and Division offices assure that the performance of the dam is adequately monitored and evaluated.

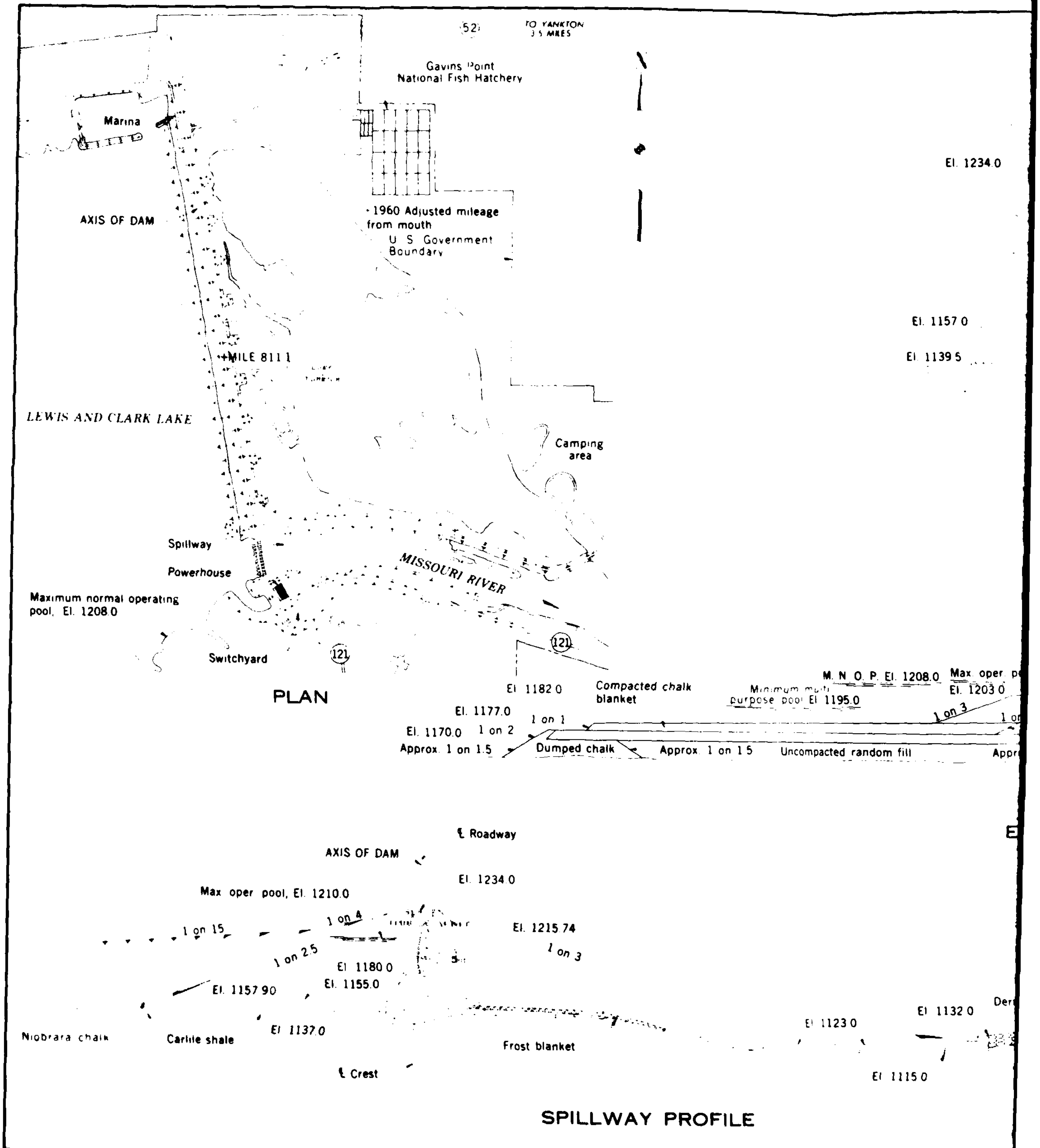
APPENDIX A
DRAWINGS


TO U.S. HWY 18
S O U T H D A K O
37
35
50
Avon
Tyndall
C M ST P & P
B O N H O M M E Y A N
Tabor
37
52
50
52
Springfield
.830
LEWIS AND CLARK LAK
.825
820.
815.
Santee
.835
Upstream limits of federal project
845
840
37
12
Niobrara
NIOBRARA RIVER
Lindy
K N O X
13
SANTEE INDIAN RESERVATION
14
Verdigre Cr
N E B R A S K
TO U.S. HWY 20



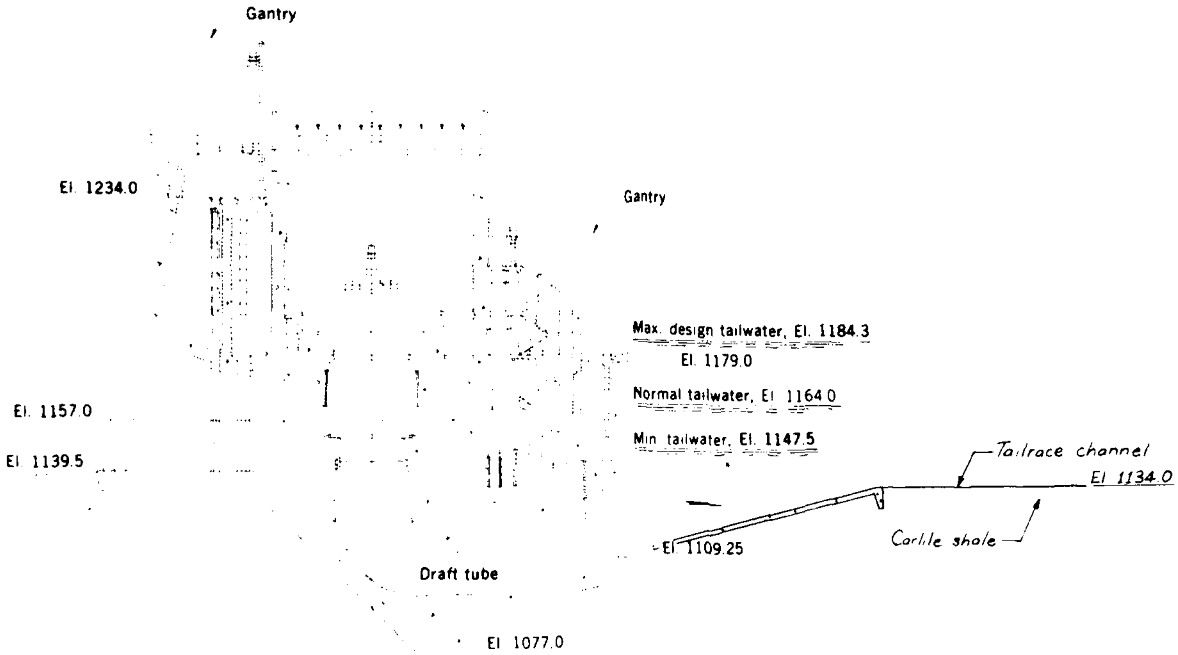
LOCATION MAP
MONTANA
NORTH DAKOTA
SOUTH DAKOTA
WYOMING
NEBRASKA
COLORADO
Fort Peck Reservoir
Lake Sakakawea
Lake Oahe
Lake Sharpe
LEWIS AND CLARK LAKE
A K O T A
Y A N K T O N
Utica
Tabor
C M St P & P
GAVINS POINT DAM
MILE 811.1 ABOVE MOUTH
YANKTON
TO VERMILION
MISSOURI RIVER
CLARK LAKE
C E D A R
Crofton
TO SIOUX CITY
O X
A S K A
TO NORFOLK
Recreation area
RESERVOIR CAPACITY 540,000 ACRE FEET (EL 1210)
TOTAL UNITED STATES LAND ACQUIRED TO DATE 34,693.0 ACRES
SCALE IN MILES
FLOOD CONTROL PROJECT
GAVINS POINT DAM
LEWIS AND CLARK LAKE
MISSOURI RIVER BASIN
NEBRASKA AND SOUTH DAKOTA
U S ARMY ENGINEER DISTRICT OMAHA
CORPS OF ENGINEERS
OMAHA NEBRASKA

2

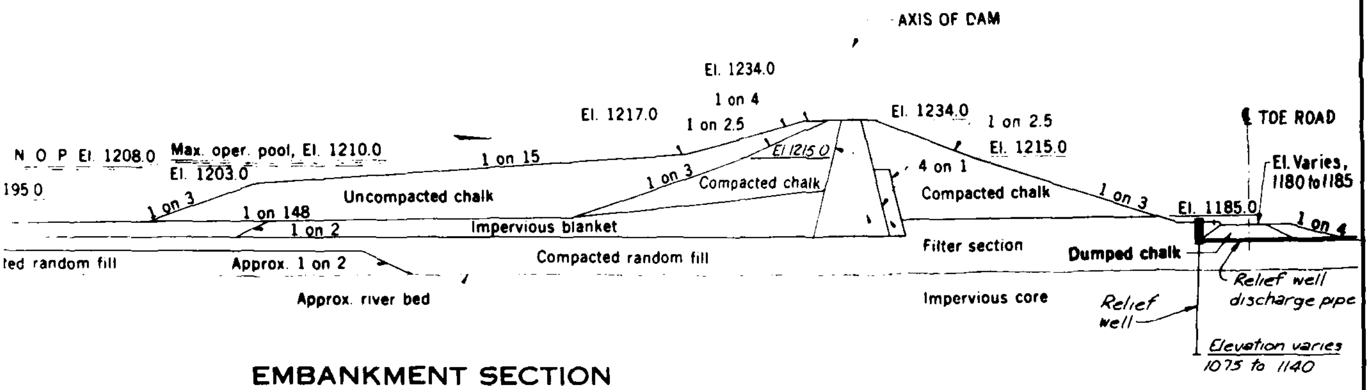


PLAN

SPILLWAY PROFILE

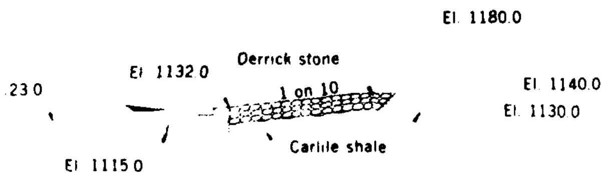


POWERHOUSE SECTION



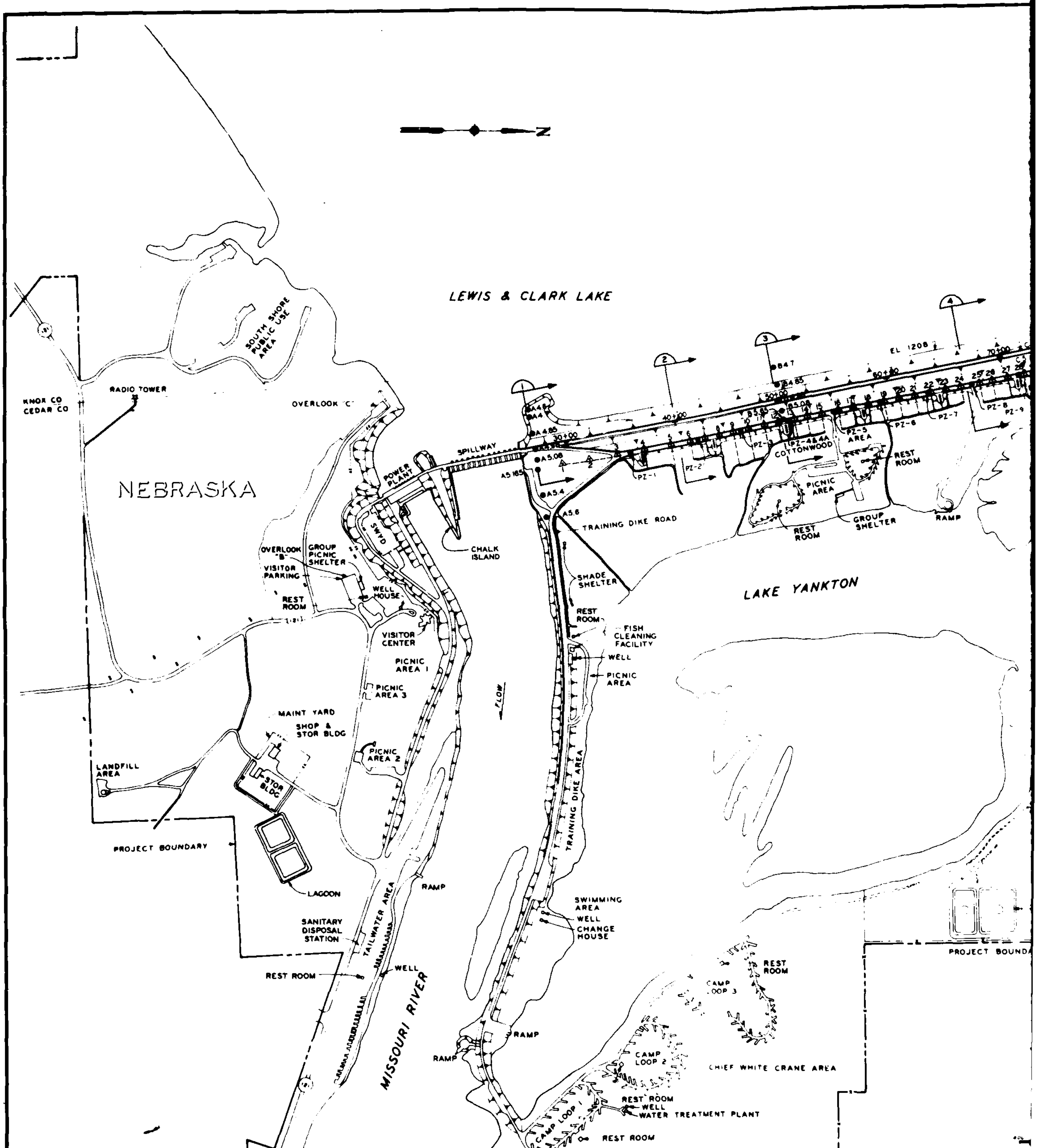
EMBANKMENT SECTION

SCALE: 1 INCH = 40 FEET
40' 0 40'

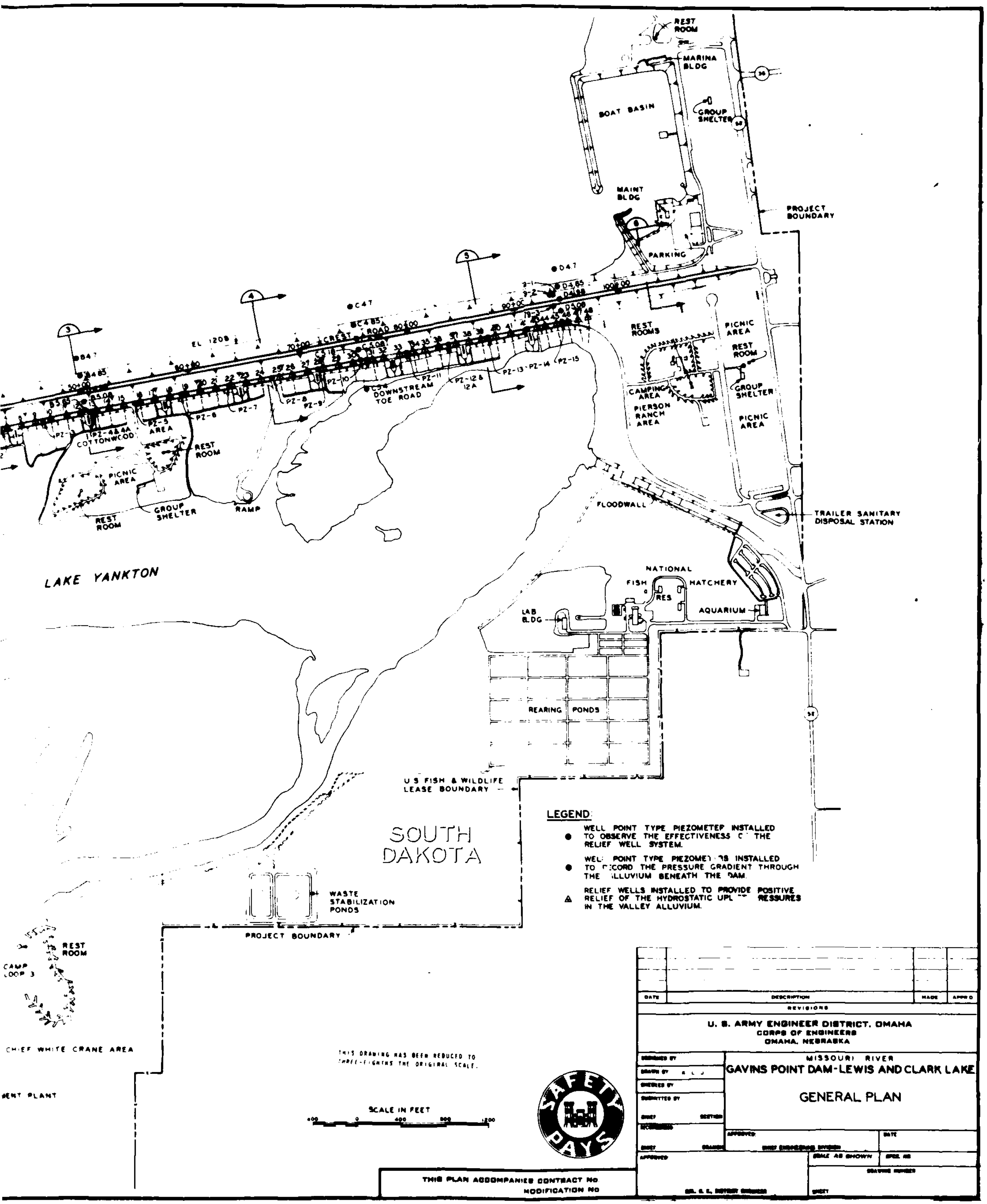


FLOOD CONTROL PROJECT
GAVINS POINT DAM
LEWIS AND CLARK LAKE
MISSOURI RIVER BASIN
NEBRASKA AND SOUTH DAKOTA

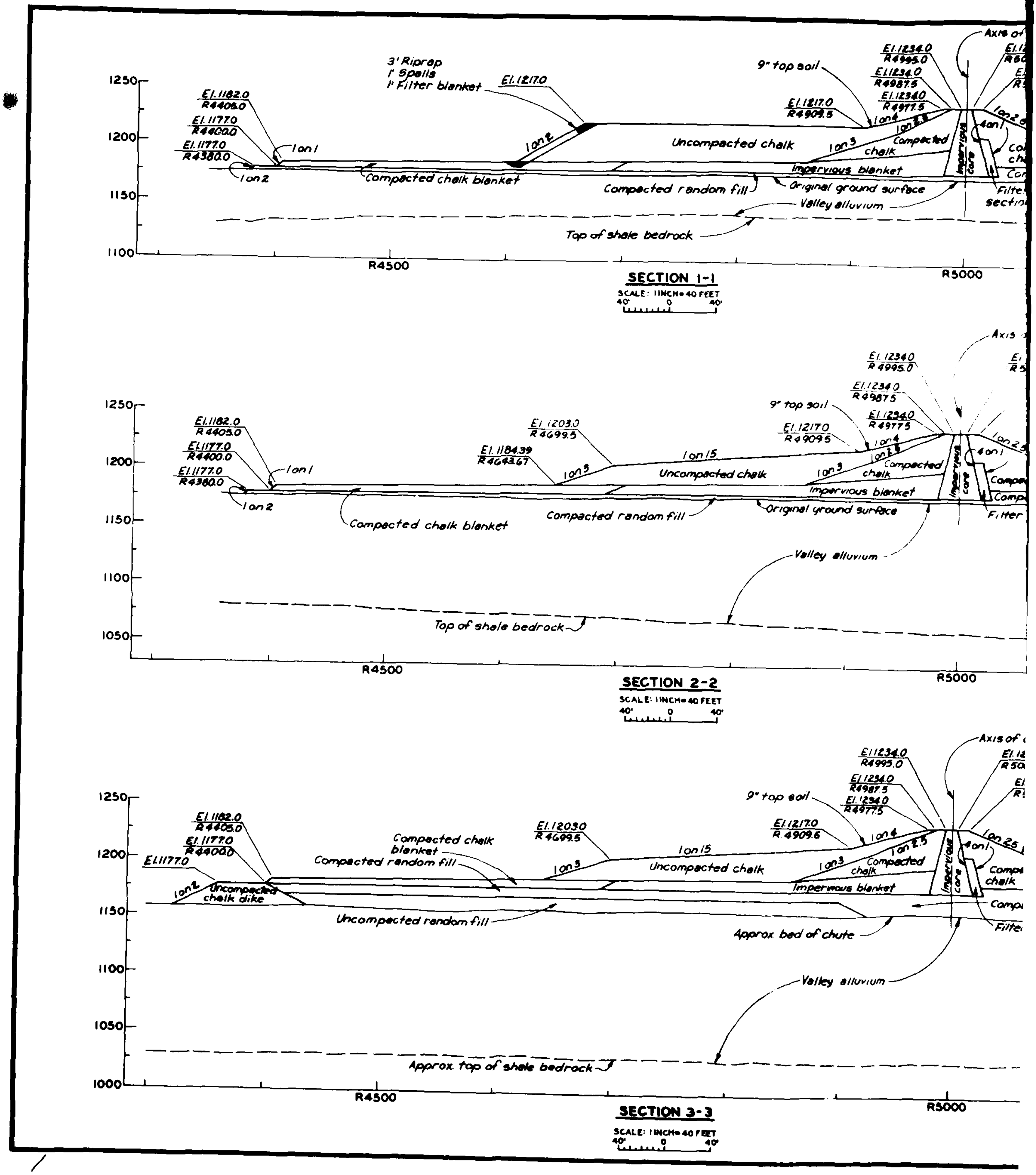
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA



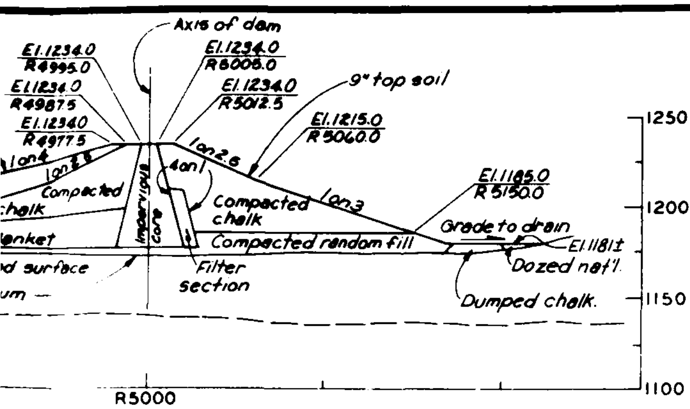
N
LEWIS & CLARK LAKE
SOUTH SHORE PUBLIC USE AREA
KNOX CO
CEDAR CO
RADIO TOWER
OVERLOOK "C"
NEBRASKA
SPILLWAY
POWER PLANT
DAMS
CHALK ISLAND
OVERLOOK "B"
GROUP PICNIC SHELTER
VISITOR PARKING
WELL HOUSE
REST ROOM
VISITOR CENTER
PICNIC AREA 1
PICNIC AREA 3
PICNIC AREA 2
MAINT YARD
SHOP & STOR BLDG
STOR BLDG
LANDFILL AREA
PROJECT BOUNDARY
LAGOON
SANITARY DISPOSAL STATION
TAILWATER AREA
RAMP
WELL
REST ROOM
MISSOURI RIVER
FLOW
TRAINING DIKE ROAD
SHADE SHELTER
REST ROOM
FISH CLEANING FACILITY
WELL
PICNIC AREA
TRAINING DIKE AREA
SWIMMING AREA
WELL
CHANGE HOUSE
RAMP
RAMP
CAMP LOOP 1
CAMP LOOP 2
CAMP LOOP 3
REST ROOM
CHIEF WHITE CRANE AREA
REST ROOM
WELL
WATER TREATMENT PLANT
REST ROOM
LAKE YANKTON
PZ-1
PZ-2
PZ-3
PZ-4&4A
COTTONWOOD
PZ-5 AREA
PZ-6
PZ-7
PZ-8
PZ-9
REST ROOM
PICNIC AREA
REST ROOM
GROUP SHELTER
RAMP
EL 1208
30+00
40+00
50+00
60+00
70+00
PROJECT BOUNDARY

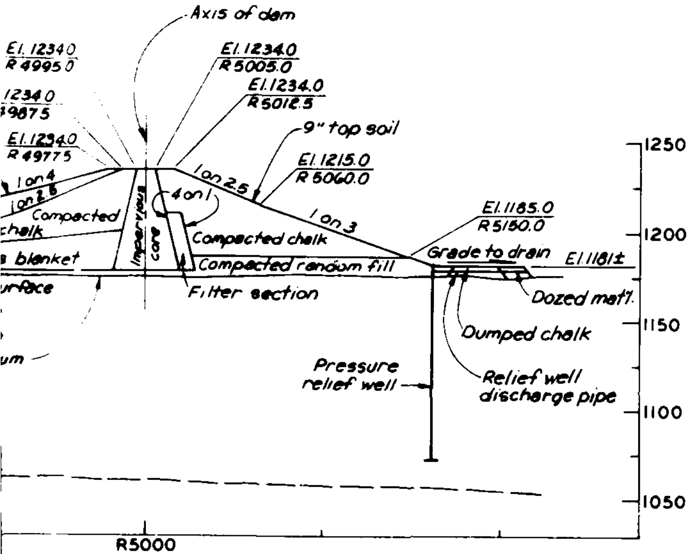

REST ROOM
MARINA BLDG
BOAT BASIN
GROUP SHELTER
MAINT BLDG
PROJECT BOUNDARY
PARKING
CREST ROAD
EL 1208
DOWNSTREAM TOE ROAD
REST ROOMS
PICNIC AREA
CAMPING AREA
PIERSON RANCH AREA
COTTONWOOD AREA
RAMP
FLOODWALL
TRAILER SANITARY DISPOSAL STATION
LAKE YANKTON
NATIONAL FISH HATCHERY
RES
LAB BLDG
AQUARIUM
REARING PONDS
U S FISH & WILDLIFE LEASE BOUNDARY
SOUTH DAKOTA
WASTE STABILIZATION PONDS
CAMP LOOP 3
CHIEF WHITE CRANE AREA
LEGEND:
WELL POINT TYPE PIEZOMETER INSTALLED TO OBSERVE THE EFFECTIVENESS OF THE RELIEF WELL SYSTEM.
WELL POINT TYPE PIEZOMETERS INSTALLED TO RECORD THE PRESSURE GRADIENT THROUGH THE ALLUVIUM BENEATH THE DAM.
RELIEF WELLS INSTALLED TO PROVIDE POSITIVE RELIEF OF THE HYDROSTATIC UPLIFT PRESSURES IN THE VALLEY ALLUVIUM.
THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.
SCALE IN FEET
400 0 400 800 1200
SAFETY PAYS
THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.
DATE
DESCRIPTION
MADE
APPR'D
REVISIONS
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA
MISSOURI RIVER
GAVINS POINT DAM-LEWIS AND CLARK LAKE
GENERAL PLAN
SCALE AS SHOWN

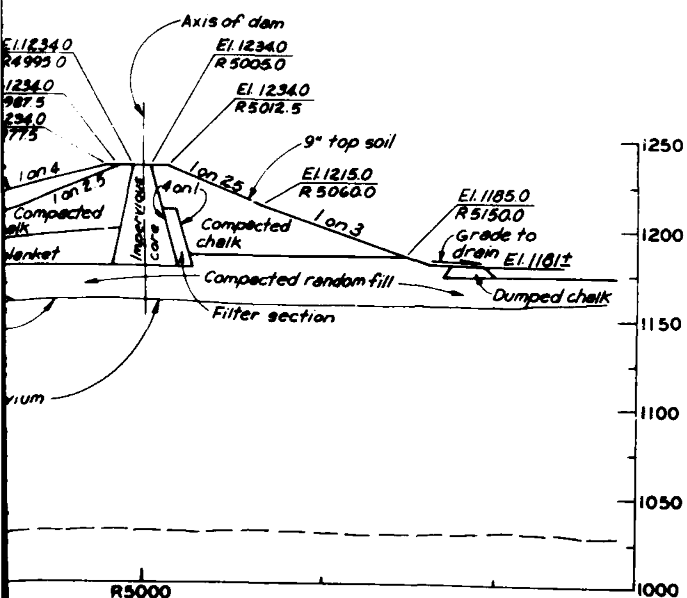
1



3' Riprap
1' Spalls
1' Filter blanket
El. 1217.0
El. 1182.0
R4405.0
El. 1177.0
R4400.0
El. 1177.0
R4380.0
1 on 1
1 on 2
Compacted chalk blanket
Uncompacted chalk
Compacted random fill
Original ground surface
Valley alluvium
Top of shale bedrock
9" top soil
Impervious blanket
Compacted chalk
Impervious core
R4500
R5000
SECTION 1-1
SCALE: 1 INCH = 40 FEET
El. 1203.0
R4699.5
El. 1184.39
R4643.67
1 on 15
SECTION 2-2
Compacted chalk blanket
Compacted random fill
Uncompacted chalk dike
Uncompacted random fill
Approx. bed of chute
Approx. top of shale bedrock
SECTION 3-3





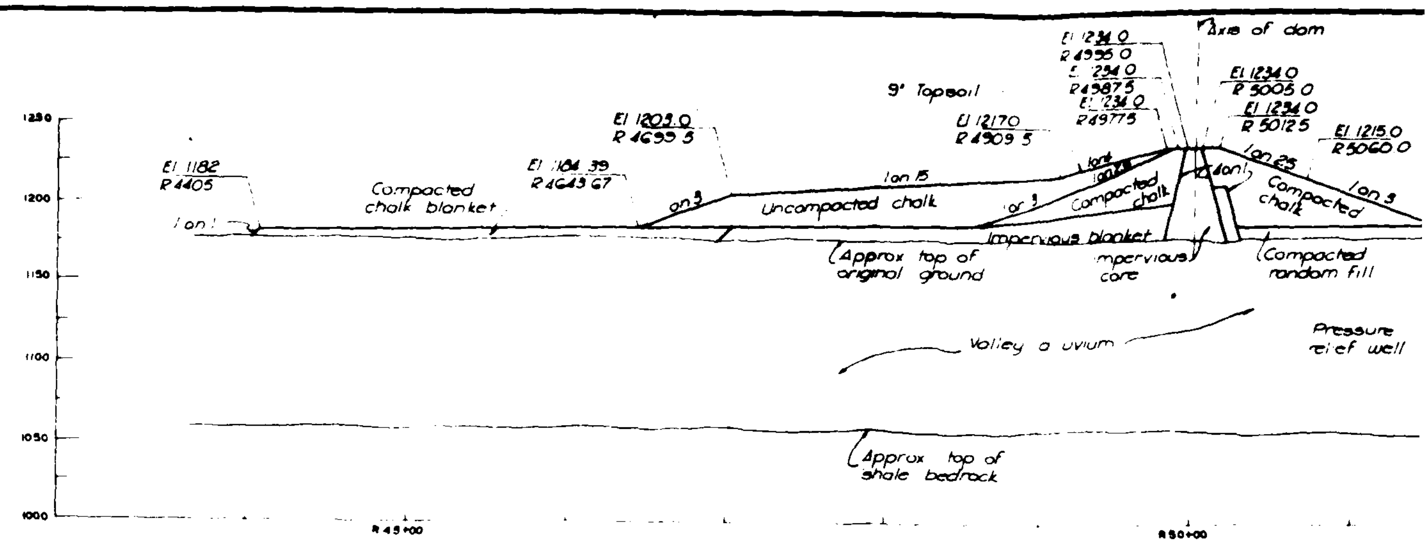


MISSOURI RIVER
GAVINS POINT DAM-LEWIS AND CLARK LAKE

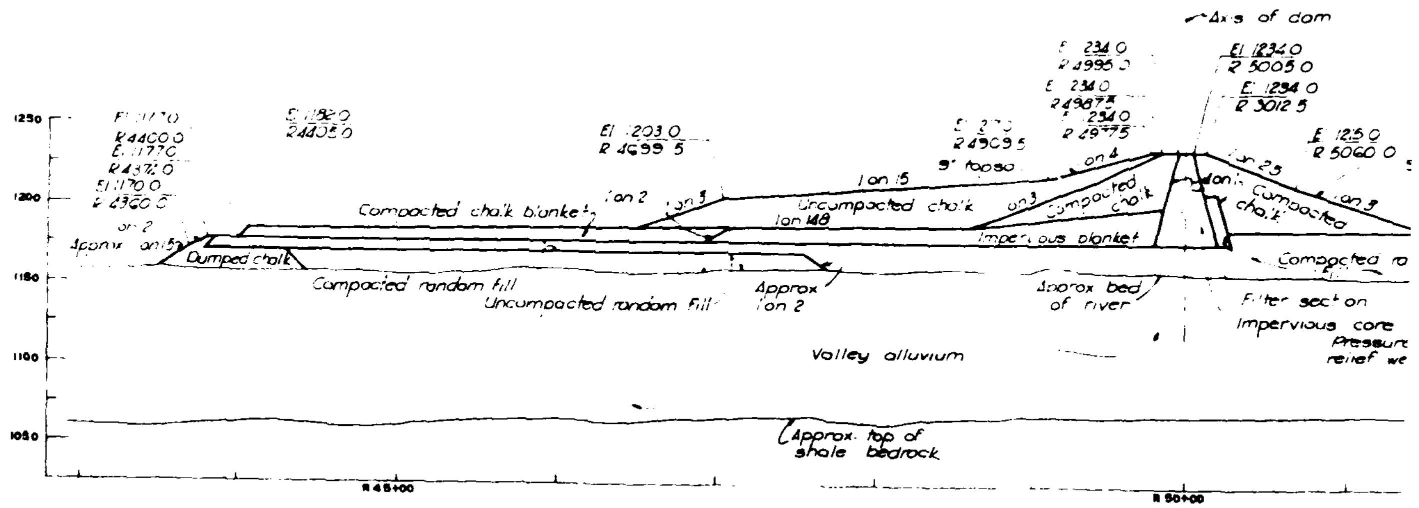
EMBANKMENT SECTIONS

SHEET 1

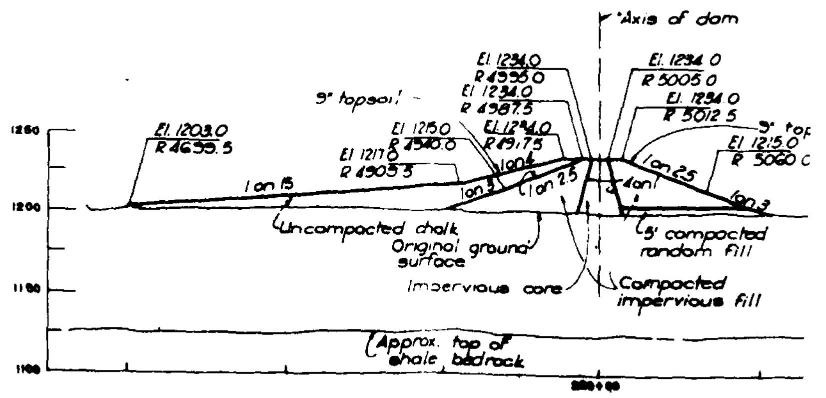
2



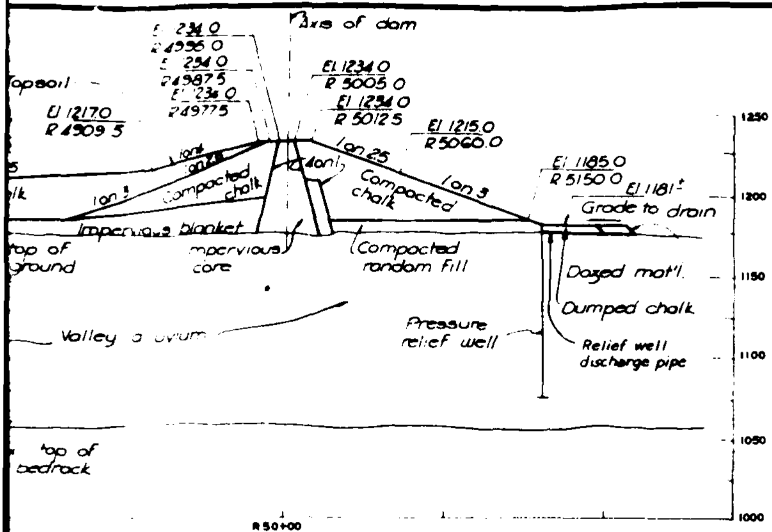
SECTION 4–4
SCALE: 1 INCH = 40 FEET

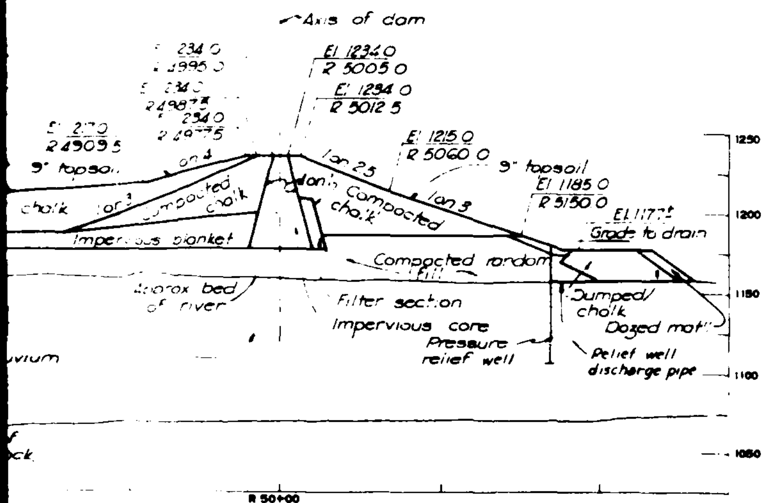


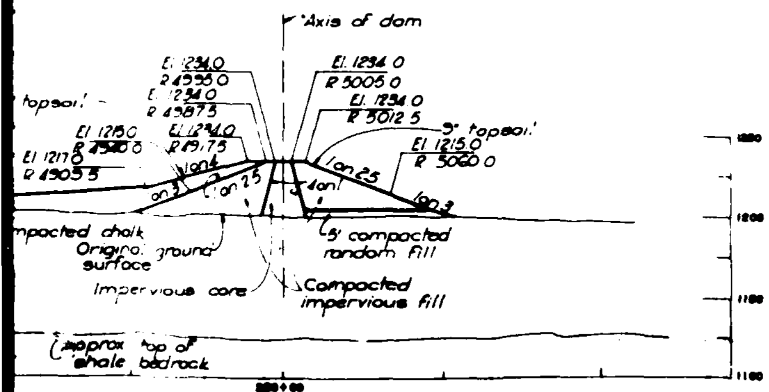
SECTION 5–5
SCALE: 1 INCH = 40 FEET



SECTION 6–6
SCALE: 1 INCH = 40 FEET







SECTION 6-6

SCALE 1 INCH = 40 FEET

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPR'D

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

GAVINS POINT DAM - LEWIS AND CLARK LAKE

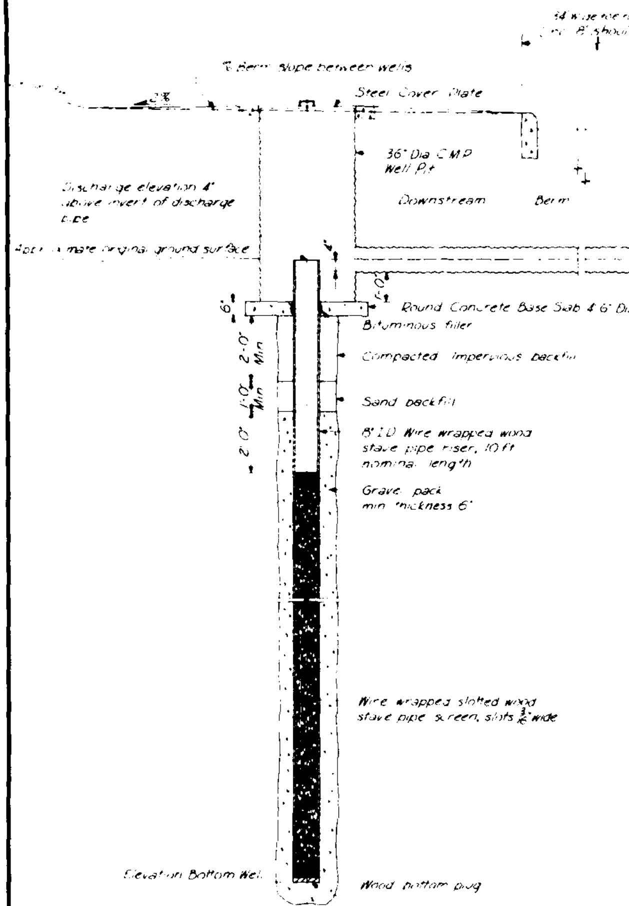
EMBANKMENT SECTIONS

SHEET 2

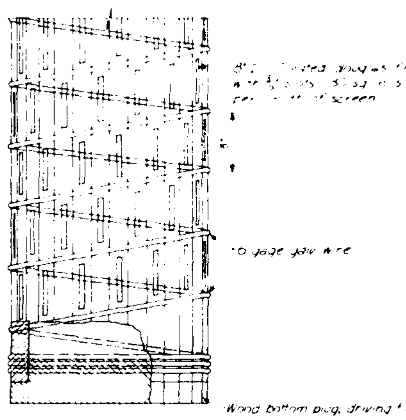
DATE: MAY 1972

THIS PLAN ACCOMPANIES CONTRACT NO.

2

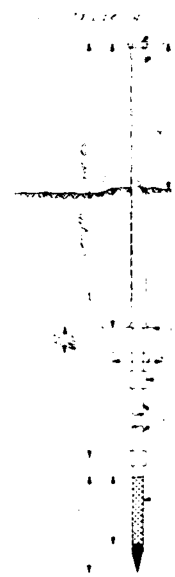


RELIEF WELL



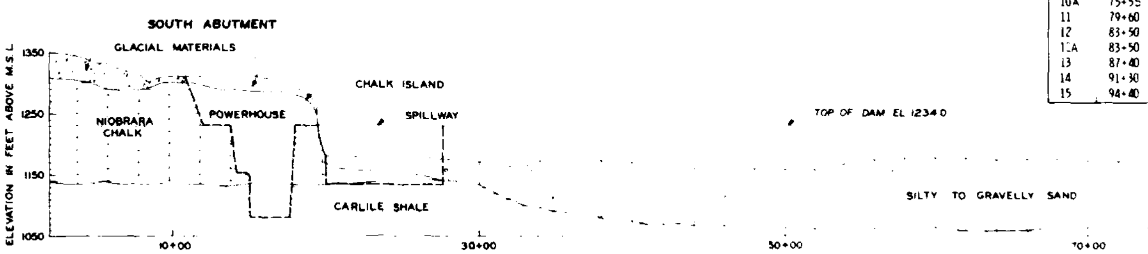
WOOD SCREEN WITH BOTTOM PLUG

TYPICAL WELL DETAILS
NO SCALE



PIEZOMETER TUE
NO SCALE

RELIEF WELL PIEZOMETE		
PIEZ. TUBE NO.	STATION	RG.
1	35+45	5156
2	41+90	5157.5
3	48+10	5162
4	50+30	5162
4A	50+30	5162
5	54+50	5160
6A	58+75	5160
7A	62+95	5160
8A	67+15	5160
9A	71+35	5160
10A	75+55	5163
11	79+60	5166
12	83+50	5166
12A	83+50	5166
13	87+40	5166
14	91+30	5170.3
15	94+40	5174

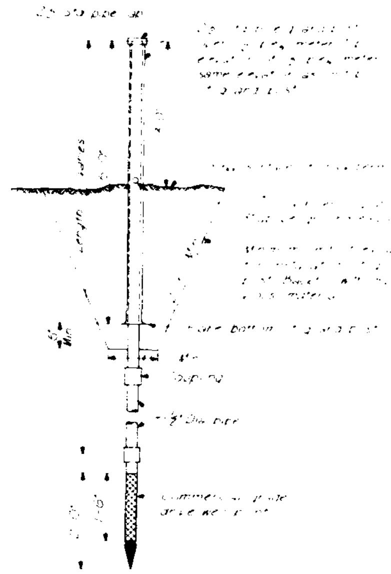


PROFILE ALONG AXIS OF DAM
SCALE 1 INCH = 400 FEET

EMBANKMENT PIEZOMETER LOCATIONS									
PIEZ. NO.	STATION	RANGE	BOTTOM EL.	TOP EL.	PIEZ. NO.	STATION	RANGE	BOTTOM EL.	TOP EL.
A 4.62	27+00	4620	1159.46	1219.69	C 4.98	75+00	4980	1158.69	1234.88
A 4.7	27+00	4700	1158.60	1219.86	C 5.08	75+00	5080	1156.18	1208.41
A 4.85	27+00	4850	1158.62	1219.75	C 5.16	75+00	5160	1156.03	1182.87
A 4.98	27+00	4980	1161.60	1236.42	C 5.4	75+00	5400	1160.05	1173.75
A 5.165	27+00	5165	1158.79	1185.89	D 4.98	95+00	4980	1160.53	1236.76
A 5.4	27+00	5400	1158.33	1181.60	D 5.18	95+00	5180	1158.38	1184.56
A 5.6	27+00	5600	1159.42	1181.72	79-1	95+00	4910	1174.23	1220.73
A' 5.165	27+80	5165	1131.68	1186.20	79-2	95+00	4910	1206.52	1220.52
B 4.98	50+00	4980	1155.80	1236.10	79-3	95+00	5010	1206.08	1236.98
B 5.08	50+00	5080	1154.30	1211.00					
B 5.165	50+00	5165	1155.27	1185.50					

PRESSURE RELIEF WELL LOCATIONS

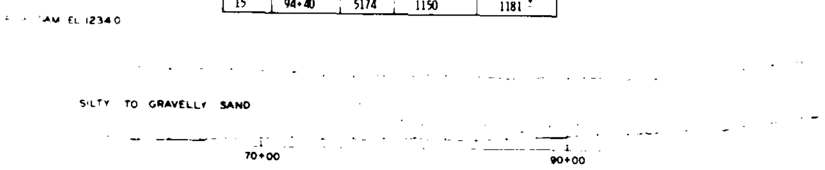
WELL NO.	STATION	RG	BOTT. EL. OF WELL	INV. EL. DISCH PIPE AT WELL PIT	EL. BERM
1	29+20	5156	1139.89	1180.60	1183 ±
2	31+70	5156	1116.08	1179.30	1183 ±
3	34+20	5156	1108.08	1178.00	1183 ±
4	36+70	5156	1102.00	1177.00	1183 ±
5	39+20	5156	1085.07	1176.70	1183 ±
6	41+20	5156	1074.83	1176.30	1183 ±
7	42+60	5159	1074.99	1174.20	1182 ±
8	44+00	5162	1074.97	1173.90	1181 ±
9	45+40	5162	1074.97	1173.60	1181 ±
10	46+80	5162	1074.17	1172.70	1181 ±
11	48+20	5162	1074.97	1173.20	1181 ±
12	49+60	5162	1074.97	1176.20	1181 ±
13	51+00	5162	1074.97	1176.20	1181 ±
14	52+40	5162	1074.97	1176.20	1182 ±
15	53+80	5160	1074.97	1176.20	1182 ±
16	55+20	5160	1074.97	1176.40	1182 ±
17	56+60	5160	1074.97	1176.20	1182 ±
18	58+00	5160	1074.97	1176.10	1182 ±
19	59+40	5160	1074.97	1176.20	1183 ±
20	60+80	5160	1074.97	1176.20	1183 ±
21	62+20	5160	1074.97	1176.10	1183 ±
22	63+60	5160	1074.97	1176.30	1183 ±
23	65+00	5160	1075.00	1176.70	1184 ±
24	66+40	5160	1074.97	1177.10	1184 ±
25	68+03	5160	1074.97	1178.10	1184 ±
26	69+20	5160	1075.00	1178.20	1184 ±
27	70+60	5160	1074.97	1177.20	1183 ±
28	72+00	5160	1084.98	1176.50	1182 ±
29	73+40	5162	1100.08	1172.30	1181 ±
30	74+30	5164	1109.97	1170.30	1181 ±
31	76+20	5166	1110.00	1169.00	1180 ±
32	77+60	5166	1110.00	1169.10	1180 ±
33	78+90	5166	1110.00	1169.10	1180 ±
34	80+20	5166	1110.00	1169.00	1180 ±
35	81+50	5166	1109.93	1169.10	1180 ±
36	82+80	5166	1128.61	1169.10	1180 ±
37	84+10	5166	1116.06	1170.10	1180 ±
38	85+40	5166	1116.77	1170.20	1180 ±
39	86+70	5166	1118.41	1170.20	1180 ±
40	88+00	5166	1116.22	1170.10	1180 ±
41	89+30	5166	1113.51	1170.70	1180 ±
42	90+60	5166	1120.53	1171.10	1181 ±
43	91+90	5166	1112.99	1172.10	1181 ±
44	92+90	5166	1115.00	1172.00	1181 ±
45	93+90	5166	1117.85	1172.10	1181 ±
46	94+90	5166	1112.97	1172.30	1181 ±
47	95+90	5166	1112.00	1172.00	1181 ±
48	96+90	5166	1112.67	1172.00	1181 ±

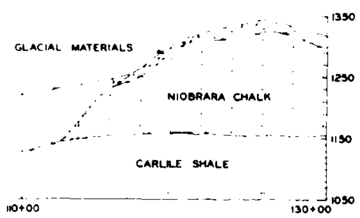


PIEZOMETER TUBE DETAIL
NO SCALE

RELIEF WELL PIEZOMETER LOCATIONS

PIEZ. TUBE NO.	STATION	RG	APPROX. BOTT. EL.	SURFACE OF BERM
1	35+45	5156	1159	1183 ±
2	41+90	5157.5	1156	1182.5 ±
3	48+10	5162	1153	1181 ±
4	50+30	5162	1153	1181 ±
4A	50+30	5162	1110	1181 ±
5	54+50	5160	1161	1182 ±
6A	58+75	5160	1161	1182 ±
7A	62+95	5160	1159	1183 ±
8A	67+15	5160	1160	1184 ±
9A	71+35	5160	1157	1182 ±
10A	75+55	5163	1161	1181 ±
11	79+60	5166	1158	1180 ±
12	83+50	5166	1156	1180 ±
12A	83+50	5166	1110	1180 ±
13	87+40	5166	1157	1180 ±
14	91+30	5170.3	1152	1181 ±
15	94+40	5174	1150	1181 ±





PROFILE ALONG AXIS OF DAM
SCALE 1 INCH = 400 FEET

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

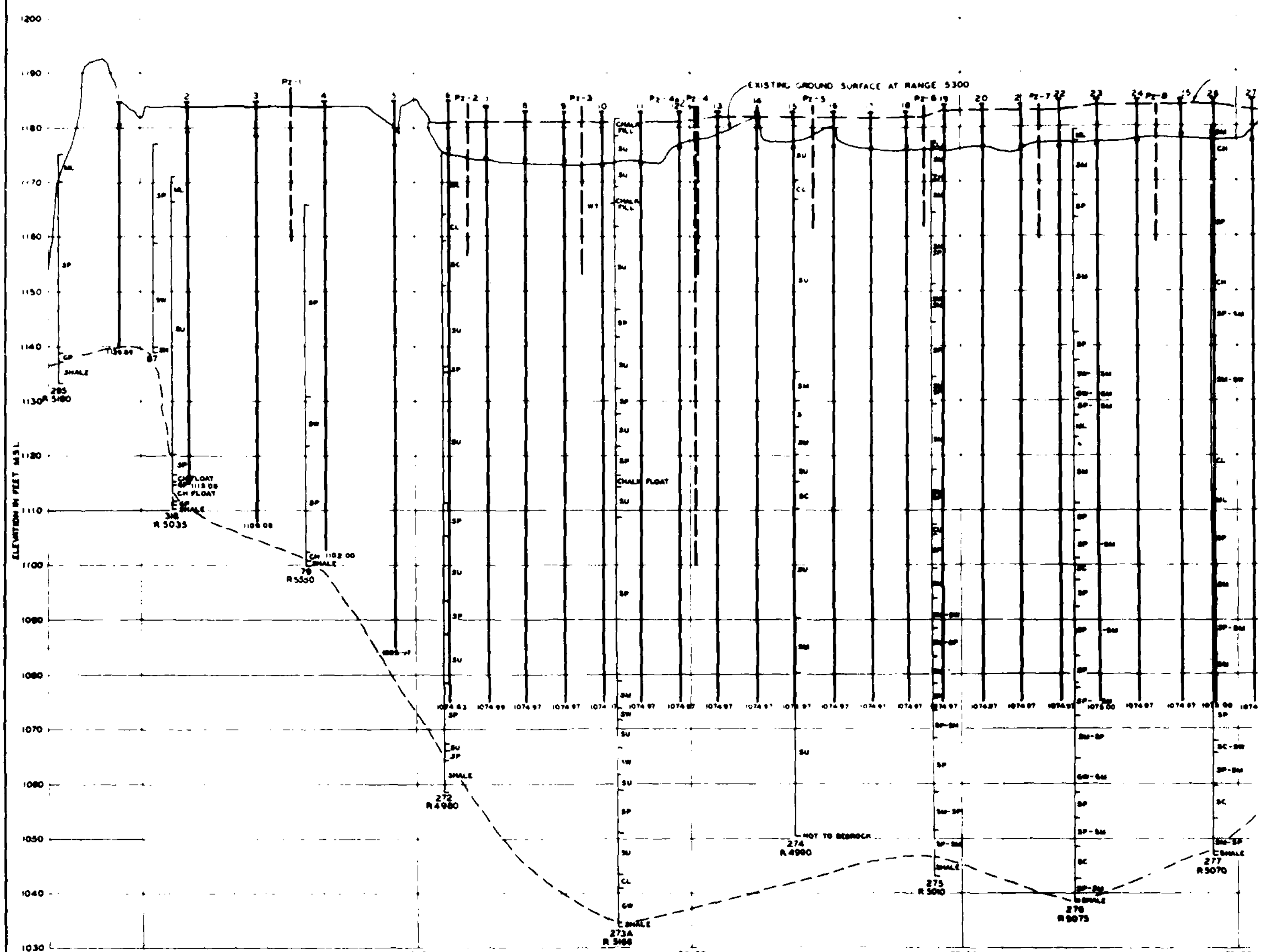
MISSOURI RIVER
GAVINS POINT DAM - LEWIS AND CLARK LAKE
UNDERSEEPAGE
RELIEF WELLS & PIEZOMETER DETAILS
GEOLOGIC PROFILE

DATE: MAY 1972

SCALE: AS SHOWN

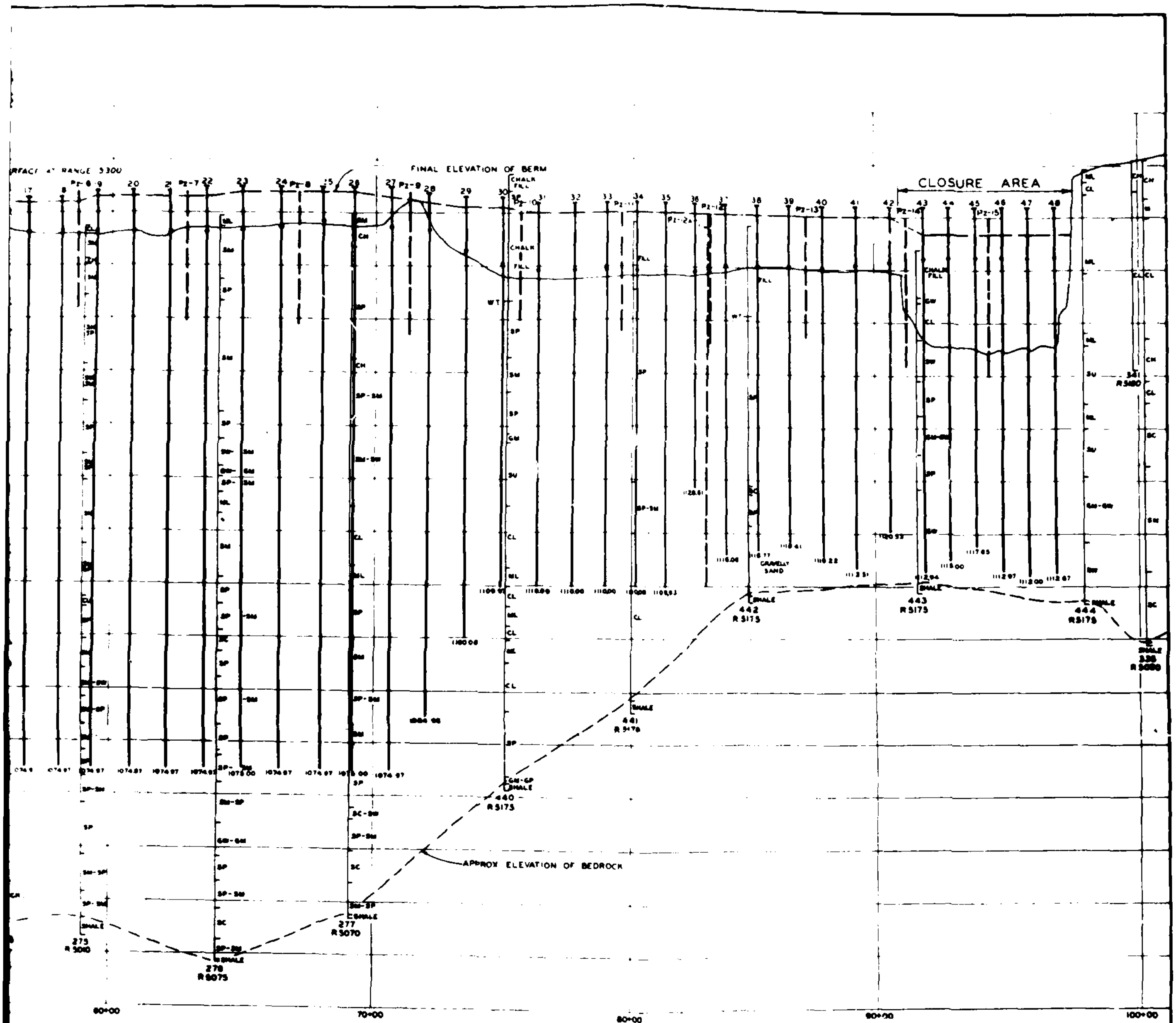
B P Pendergrass

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.



SOIL CLASSIFICATION LEGEN

- GW GRAVEL OR SANDY GRAVEL WELL-G
- GP GRAVEL OR SANDY GRAVEL POORL
- GM SILTY GRAVEL OR SILTY SANDY G
- GC CLAYEY GRAVEL OR CLAYEY SANDY
- SW SAND OR GRAVELLY SAND WELL-
- SU SAND UNIFORMLY-GRADED
- SP SAND OR GRAVELLY SAND POORL
- SM SILTY SAND OR SILTY GRAVELLY S
- SC CLAYEY SAND OR CLAYEY GRAVELL
- ML SILTS SANDY SILTS GRAVELLY SI
- CL LEAN CLAYS SANDY CLAYS OR GR
- OL ORGANIC SILTS OR LEAN ORGANI
- CH FAT CLAYS
- WT WATER TABLE PRIOR TO CLOSU
- 000 HOLE NUMBER



SOIL CLASSIFICATION LEGEND

- GW GRAVEL OR SANDY GRAVEL WELL-GRADED
- GP GRAVEL OR SANDY GRAVEL POORLY GRADED
- GM SILTY GRAVEL OR SILTY SANDY GRAVEL
- GC CLAYEY GRAVEL OR CLAYEY SANDY GRAVEL
- SW SAND OR GRAVELLY SAND WELL-GRADED
- SU SAND UNIFORMLY-GRADED
- SP SAND OR GRAVELLY SAND POORLY-GRADED
- SM SILTY SAND OR SILTY GRAVELLY SAND
- SC CLAYEY SAND OR CLAYEY GRAVELLY SAND
- ML SILTS SANDY SILTS GRAVELLY SILTS
- CL LEAN CLAYS SANDY CLAYS OR GRAVELLY CLAYS
- OL ORGANIC SILTS OR LEAN ORGANIC CLAYS
- CH FAT CLAYS
- WT WATER TABLE PRIOR TO CLOSURE
- 000 HOLE NUMBER

LEGEND

22 TOP OF WOOD RISER
RELIEF WELL

Pz-3 PIEZOMETER TUBE

DATE	DESCRIPTION	MADE	APPR'D

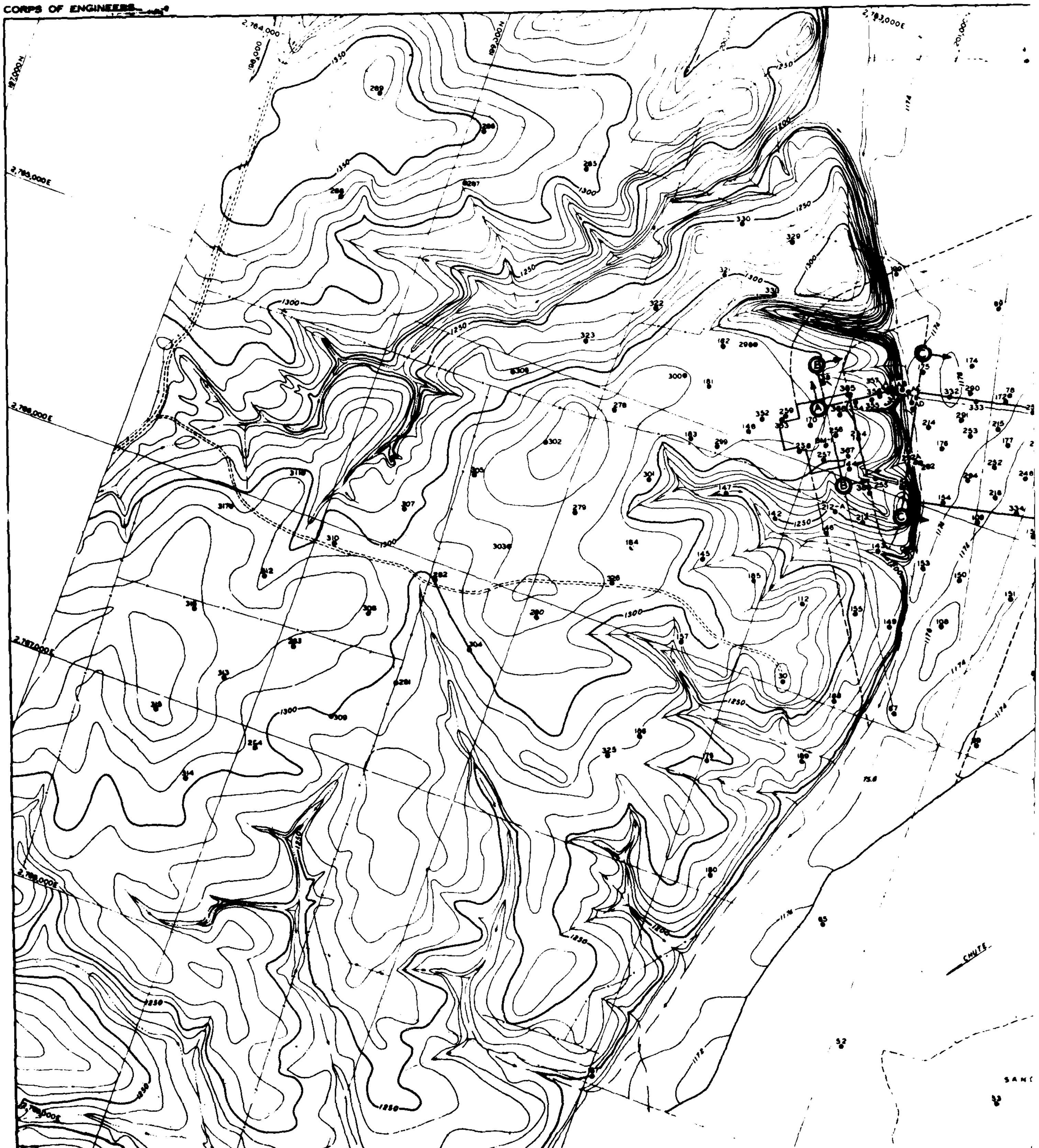
REVISIONS

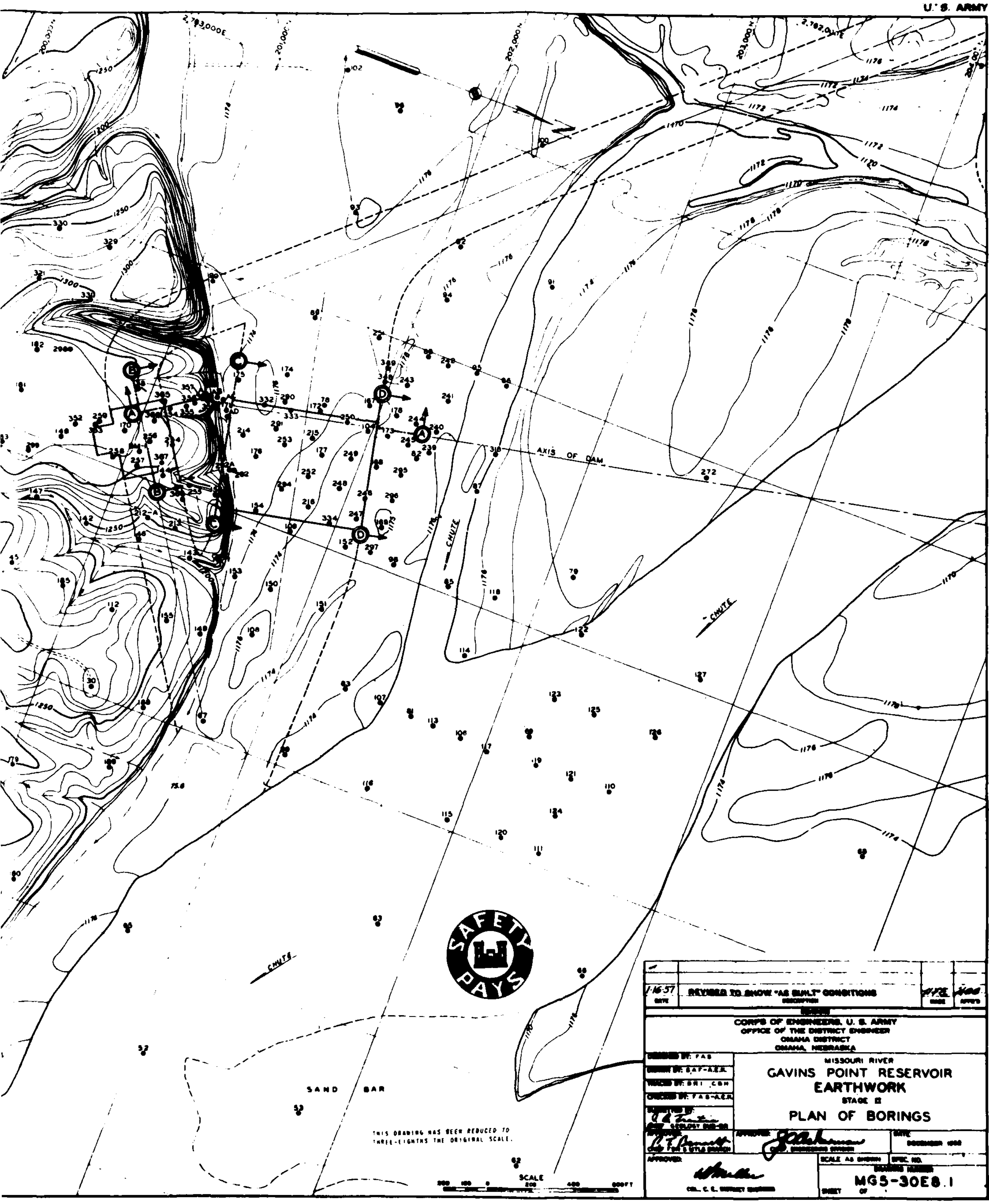
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT DAM-LEWIS AND CLARK LAKE
UNDERSEEPAGE
PROFILE OF WELLS
AND PIEZOMETERS

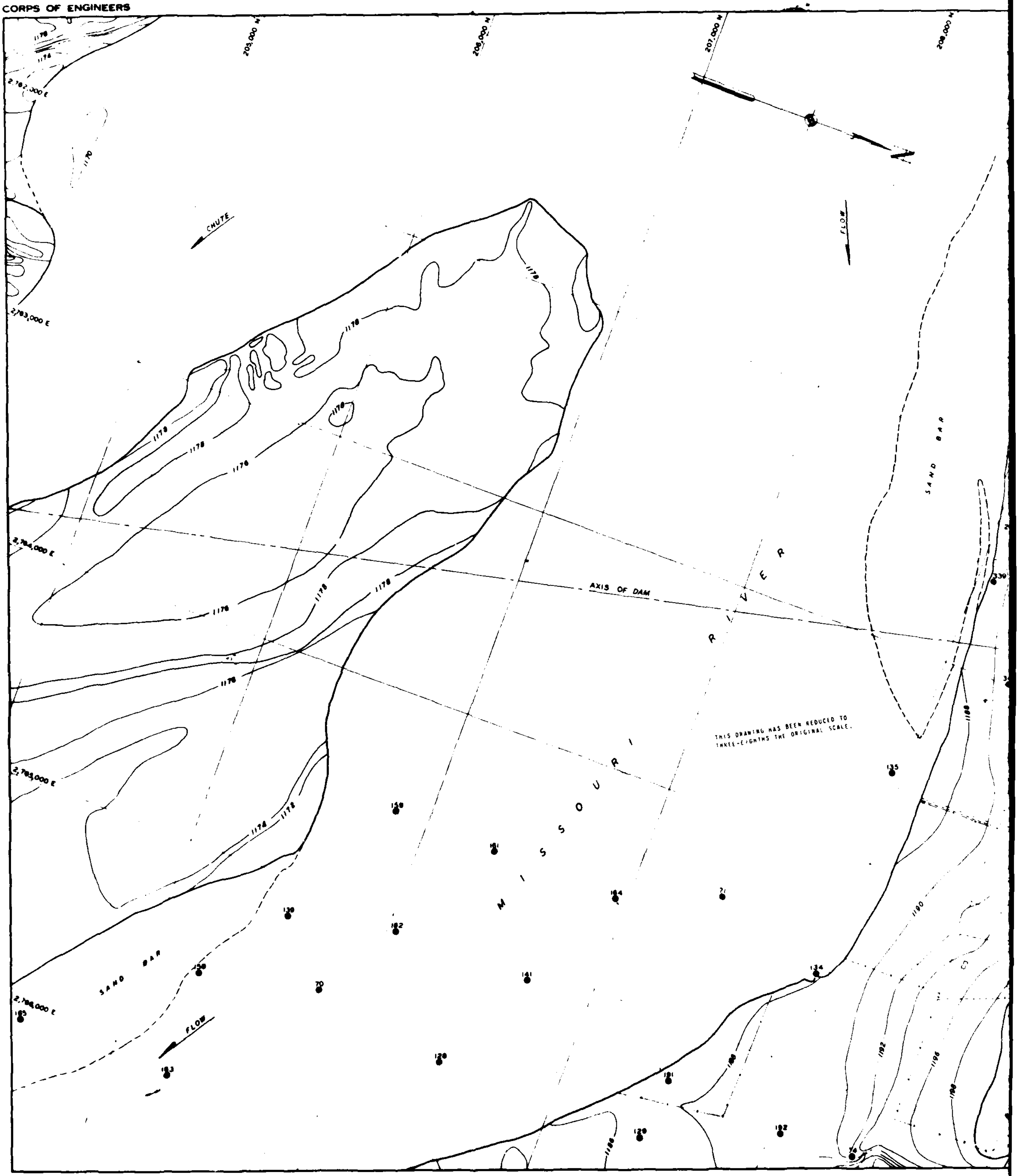
DESIGNED BY E O N
DRAWN BY T O F
TRACED BY
CHECKED BY L O N
SUBMITTED BY: CHIEF, FDN & SS SEC
RECOMMENDED: CHIEF, FDN & MATLS BR
APPROVED: CHIEF, ENGINEERING DIVISION
DATE: MAY 1972
APPROVED: COL. C E DISTRICT ENGINEER
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET OF

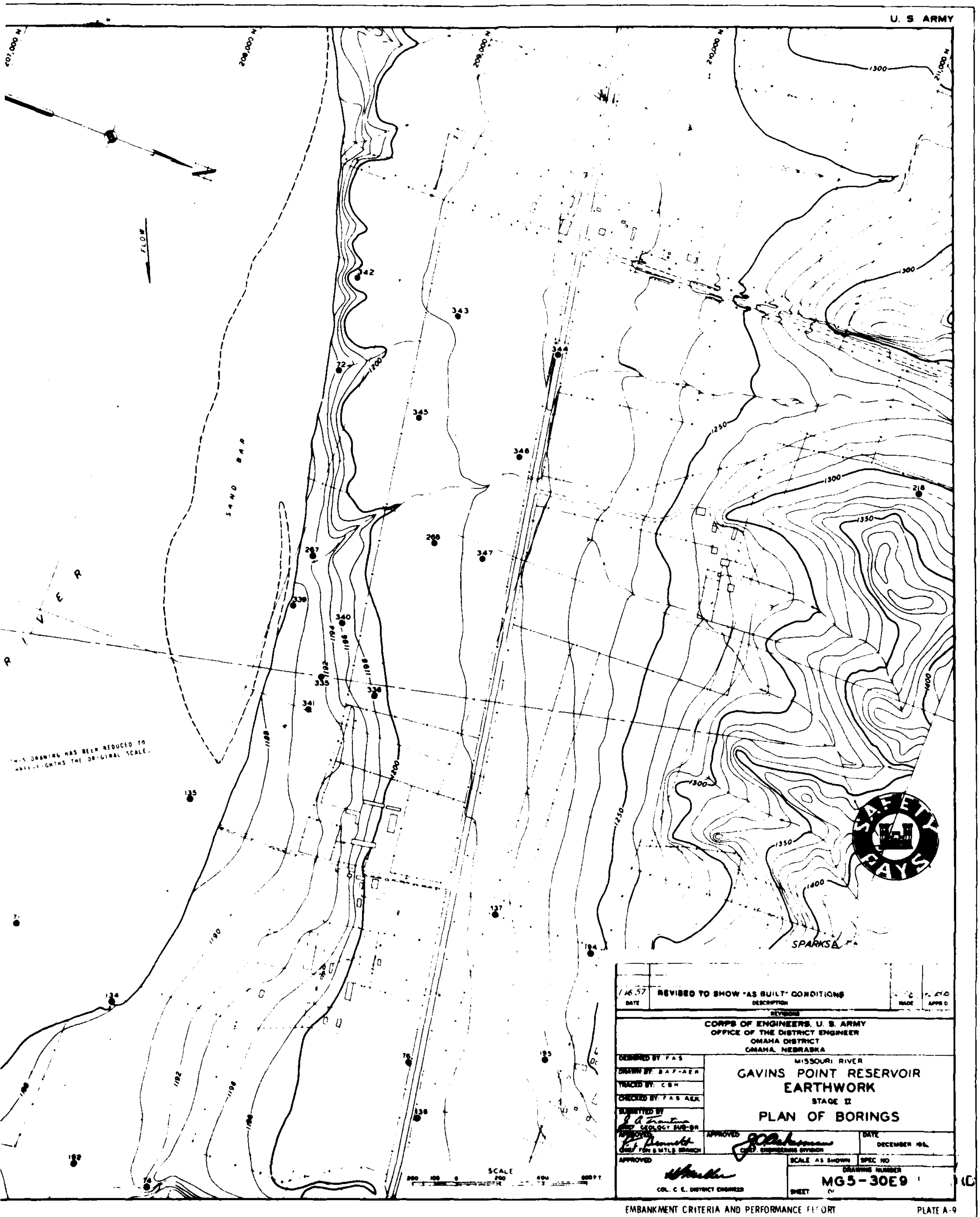
2


CORPS OF ENGINEERS
CHUTE

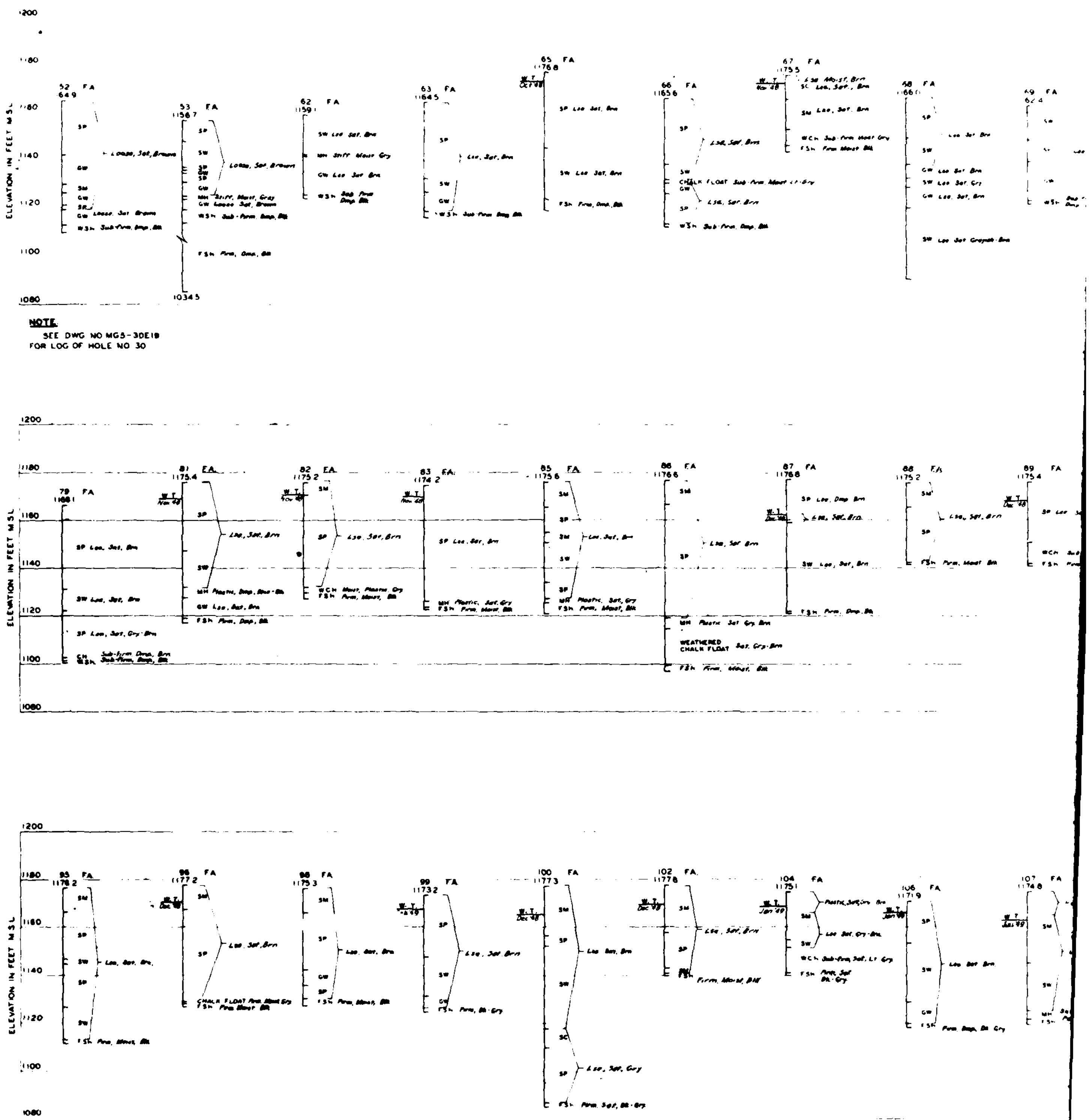

U. S. ARMY
AXIS OF DAM
CHUTE
SAND BAR
THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.
SCALE
1-16-57
REVISED TO SHOW "AS BUILT" CONDITIONS
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
PLAN OF BORINGS
SCALE AS SHOWN
SPEC. NO.
MGS-30E8.1

CORPS OF ENGINEERS

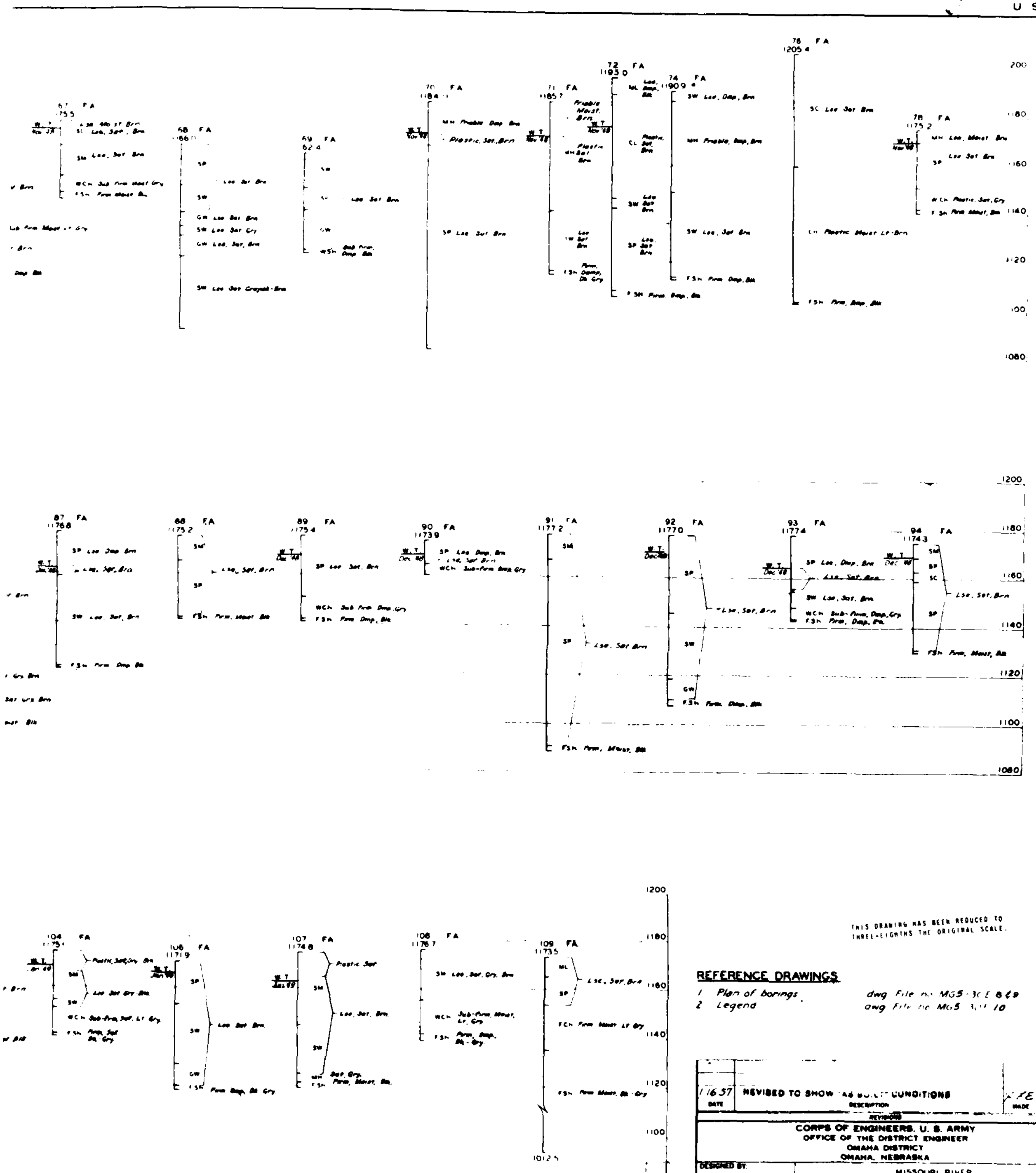



U. S. ARMY
207,000 N
208,000 N
209,000 N
210,000 N
211,000 N
FLOW
SAND BAR
R I V E R
1300
1250
1200
1350
1400
1188
1190
1192
1194
1196
1198
342
343
344
345
346
72
218
298
297
347
339
340
335
336
341
135
137
194
134
76
195
338
182
74
THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE.
SPARKS
SCALE
SAFETY PAYS
REVISED TO SHOW "AS BUILT" CONDITIONS
DATE
DESCRIPTION
MADE
APPR'D
REVISIONS
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
DESIGNED BY F A S
DRAWN BY D A F - A E R
TRACED BY C S H
CHECKED BY F A S A E R
SUBMITTED BY
GEOLOGY SUB-BR
CHIEF FDN & MTLS BRANCH
APPROVED
CHIEF, ENGINEERING DIVISION
COL. C. E. DISTRICT ENGINEER
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
PLAN OF BORINGS
DATE
DECEMBER 195
SCALE AS SHOWN
SPEC NO
DRAWING NUMBER
MG5-30E9
SHEET

CORPS OF ENGINEERS



U S ARMY



THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

REFERENCE DRAWINGS

1 Plan of borings — dwg File no MGS-30E 8 & 9
2 Legend — dwg File no MGS 30E 10

DATE	DESCRIPTION	MADE	APPR'D
1 16 57	REVISED TO SHOW "AS BUILT" CONDITIONS	FE	DR

REVISIONS

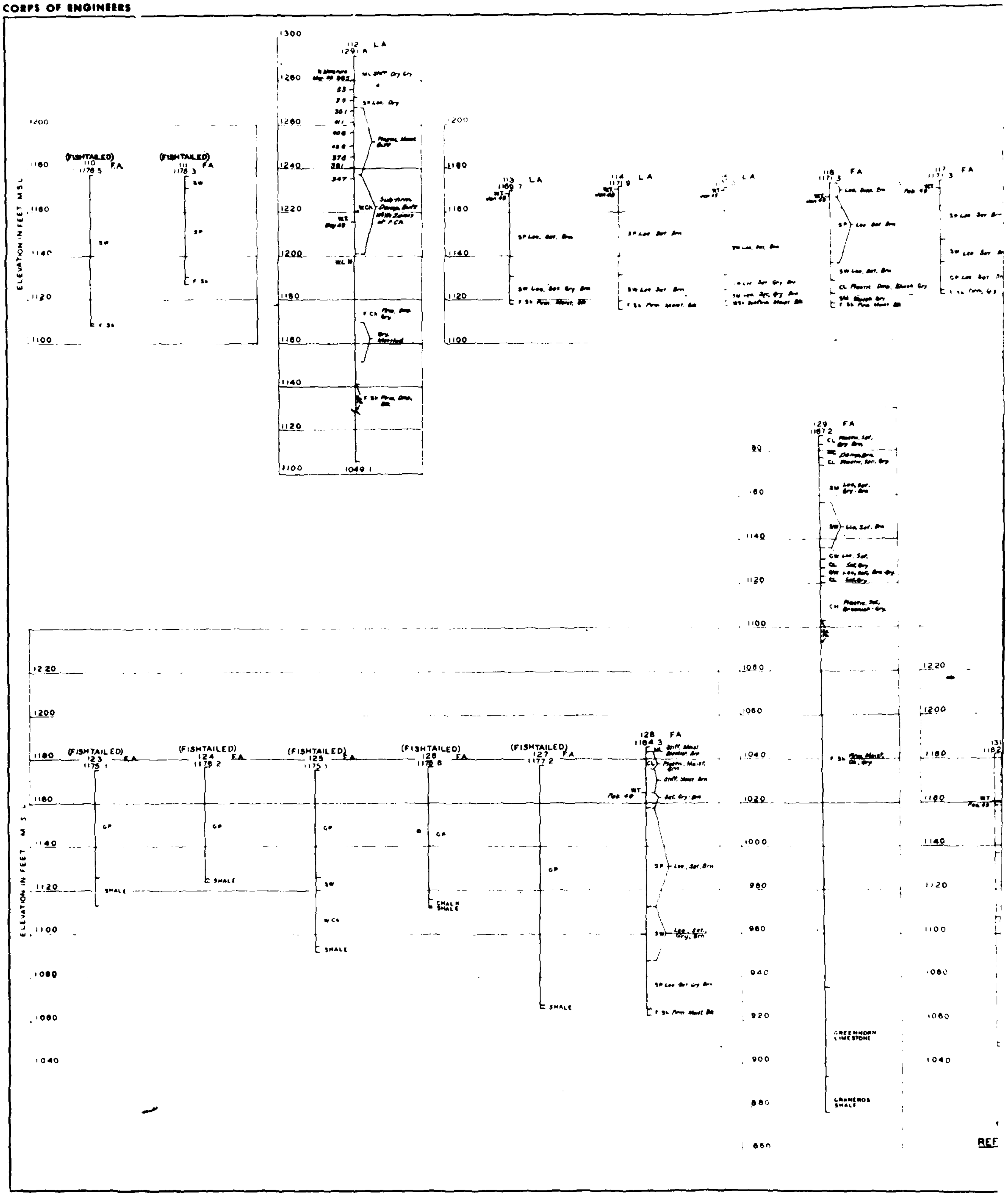
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

DESIGNED BY:
DRAWN BY: E R C
TRACED BY: J E D
CHECKED BY: A E R
SUBMITTED BY: CHIEF GEOLOGY SUB-SEC.
APPROVED: CHIEF, FDN & MTLS BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
APPROVED: COL., C. E. DISTRICT ENGINEER

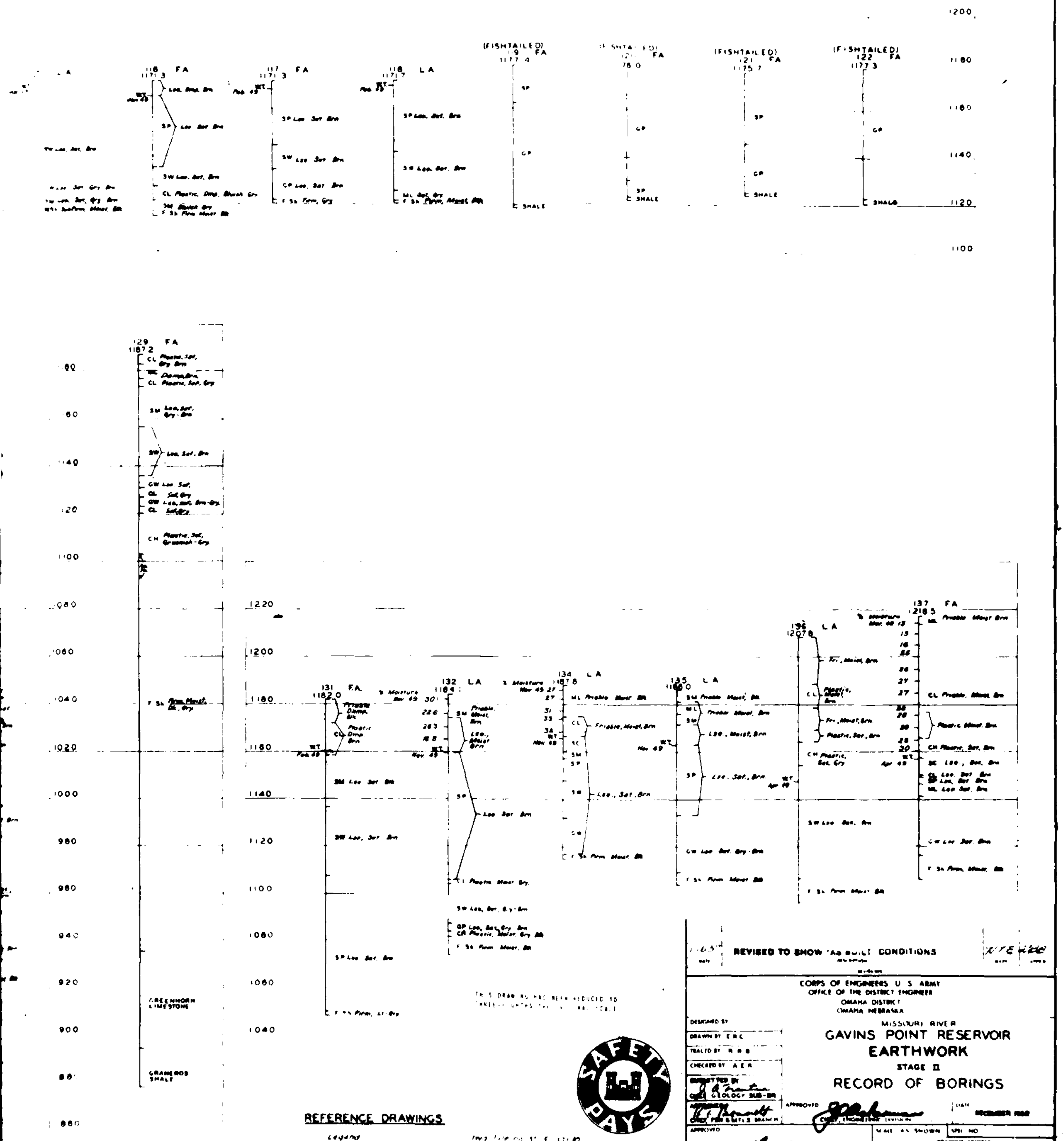
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
RECORD OF BORINGS

DATE: DECEMBER 1952
SCALE: AS SHOWN
SPEC NO
DRAWING NUMBER: MGS-30E14.1
SHEET OF

CORPS OF ENGINEERS



U. S. ARMY



REVISED TO SHOW "AS BUILT" CONDITIONS

CORPS OF ENGINEERS U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA NEBRASKA

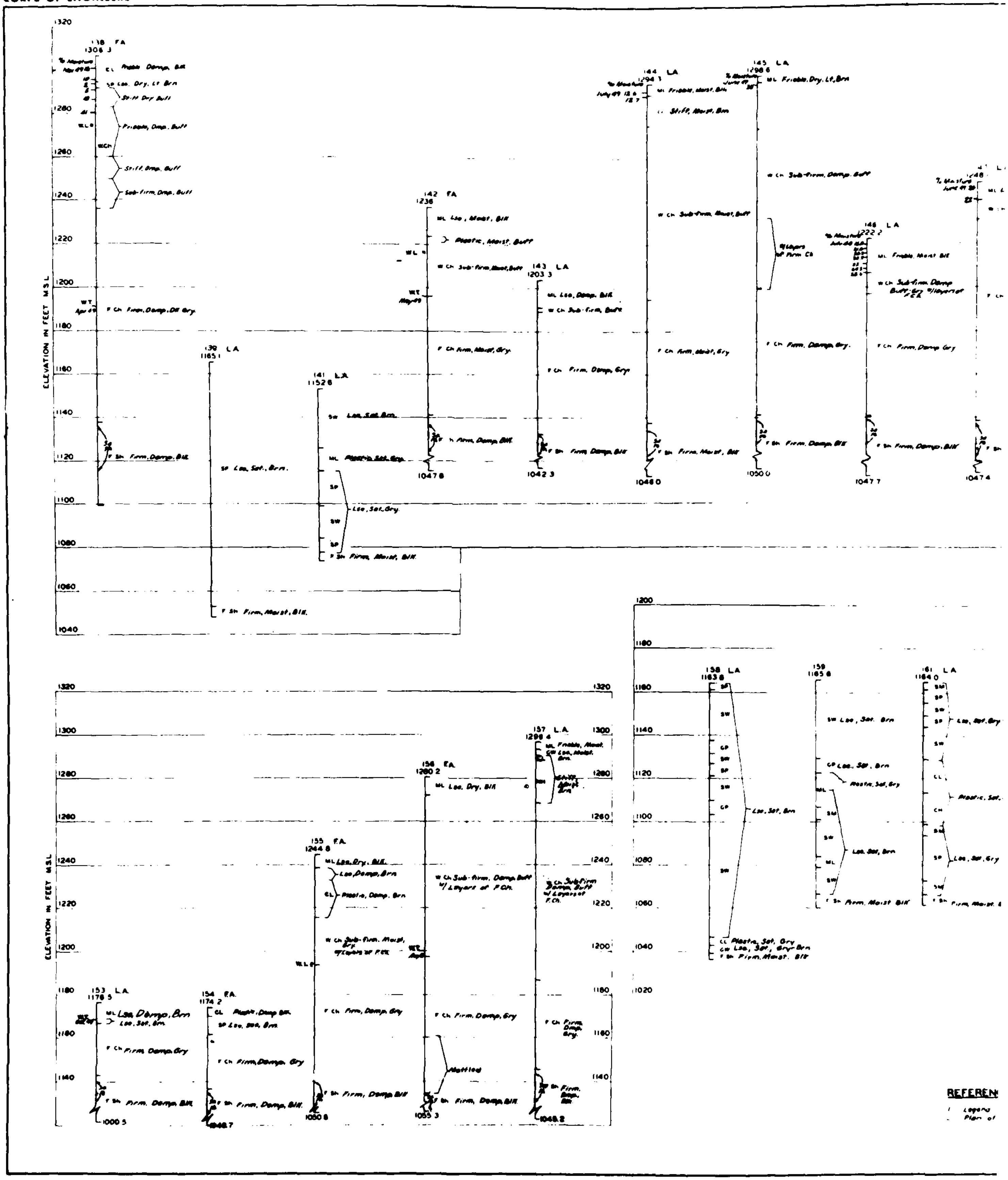
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
RECORD OF BORINGS

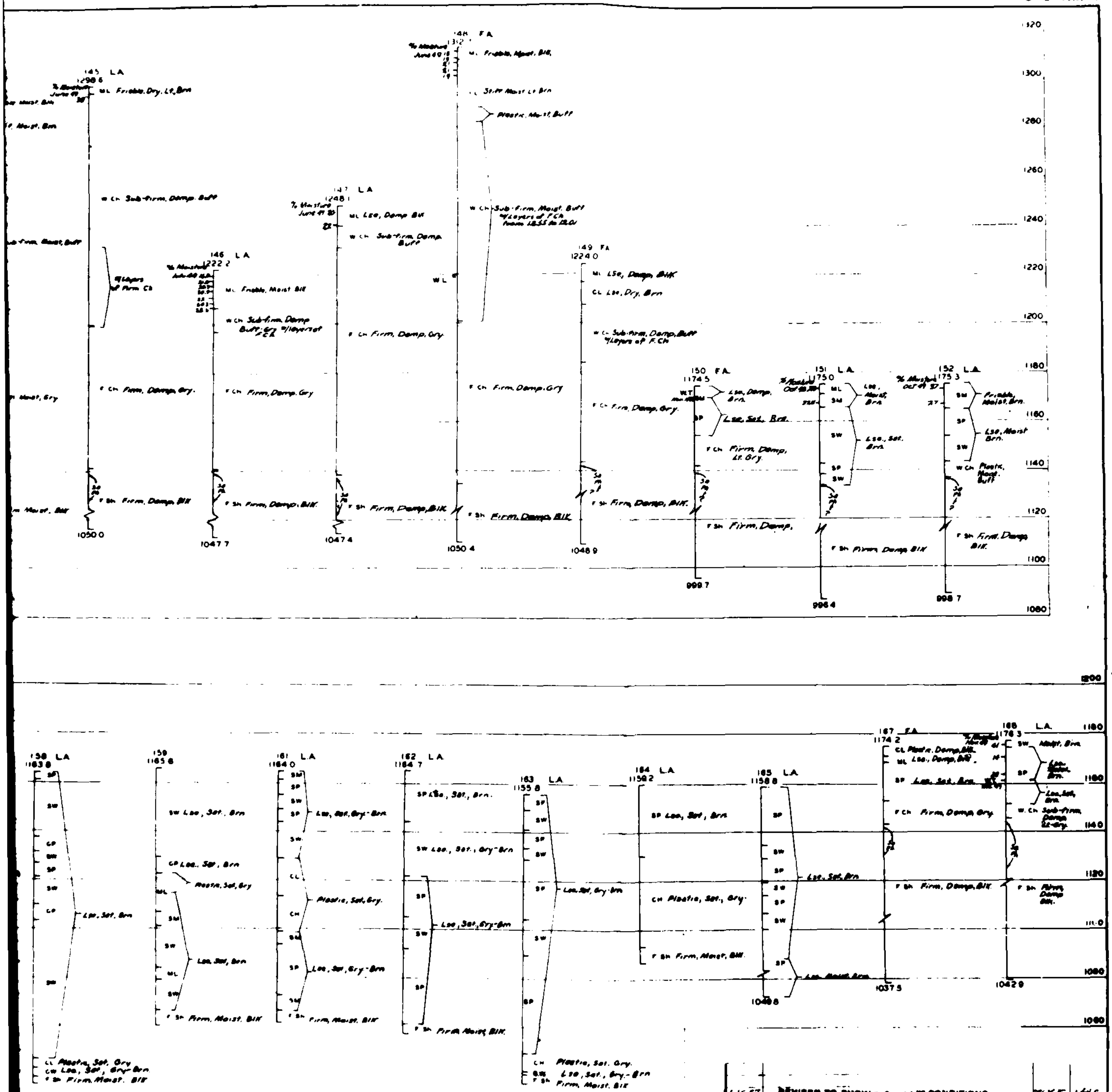
DRAWN BY E.R.C.
TRACED BY R.R.B.
CHECKED BY A.E.R.
SUBMITTED BY
CHIEF, GEOLOGY SUB-BR
APPROVED
SCALE AS SHOWN

REFERENCE DRAWINGS
Legend
Plan of borings

MG5-30E15.1

CORPS OF ENGINEERS





THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

SAFETY PAYS

REFERENCE DRAWINGS

1. Legend — Dwg. File No. MG5 101/10
2. Plan of borings — Dwg. File No. MG5 101/889

11/6/57	REVISED TO SHOW "AS BUILT" CONDITIONS	A.F.E.

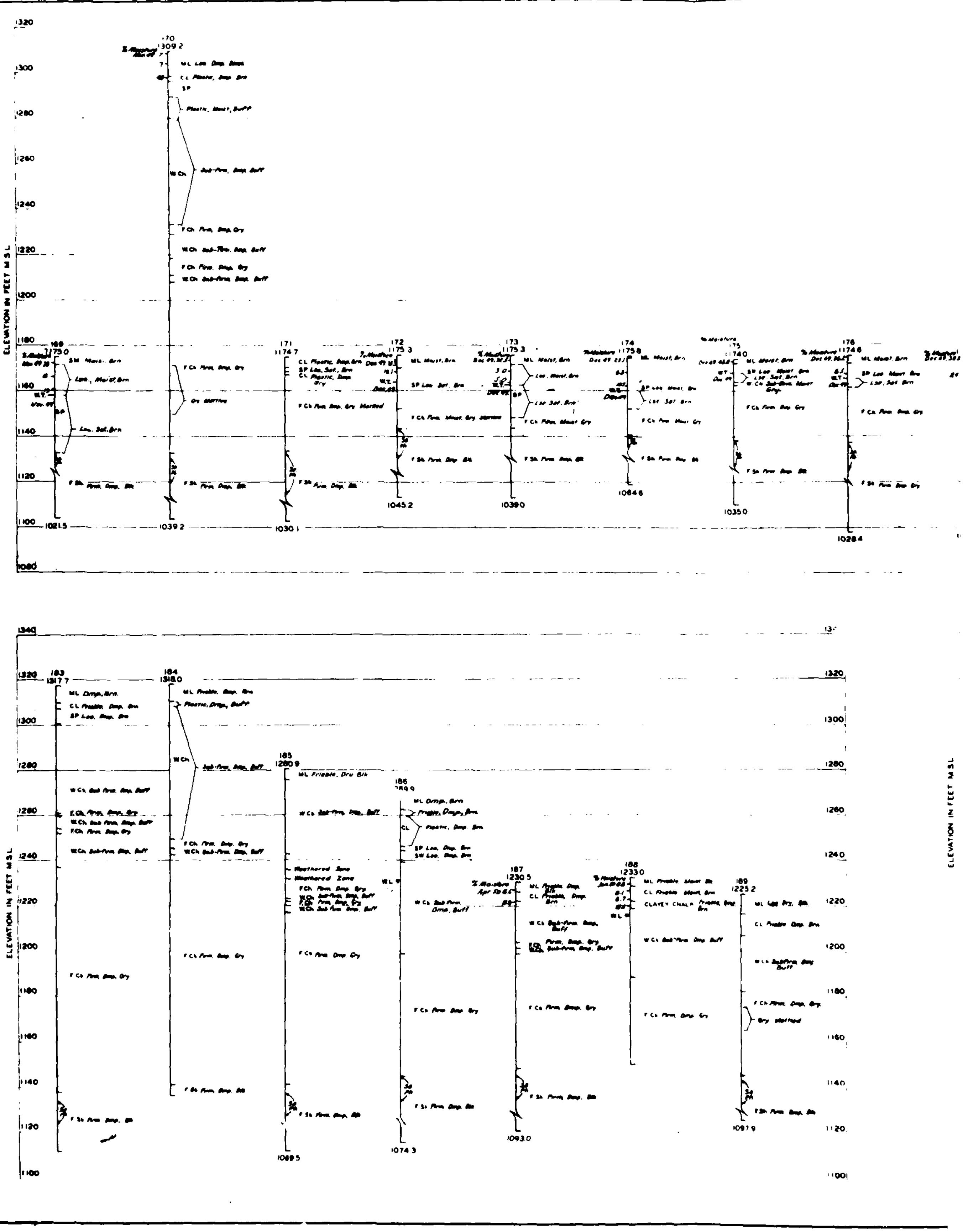
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

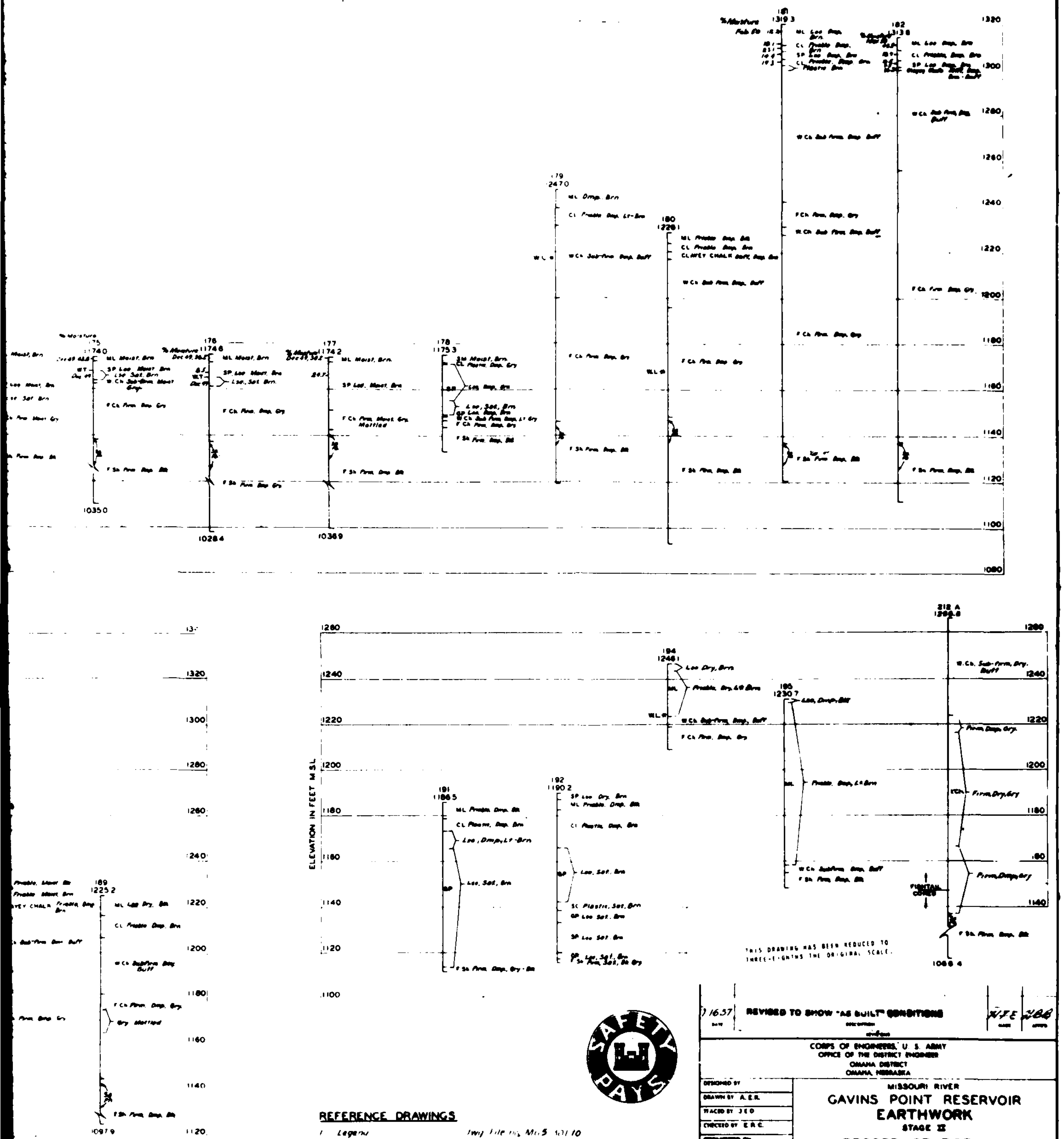
MISSOURI RIVER

GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
RECORD OF BORINGS

DESIGNED BY
DRAWN BY: E.R.C.
TRACED BY: D.M.H.
CHECKED BY: A.E.H.
SUBMITTED BY: CHIEF, GEOLOGY SUB-SECTION
APPROVED: CHIEF, SOILS & GEOLOGY BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
APPROVED: COL., C. E., DISTRICT ENGINEER

DATE: DECEMBER 1955
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER: MG5-30E16.1


ELEVATION IN FEET M.S.L.
1320
1300
1280
1260
1240
1220
1200
1180
1160
1140
1120
1100
1080
170
1309.2
169
1175.0
171
1174.7
172
1175.3
173
1175.3
174
1175.8
175
1174.0
176
1174.6
1021.5
1039.2
1030.1
1045.2
1039.0
1064.6
1035.0
1028.4
1340
183
1317.7
184
1318.0
185
1280.9
186
187
1230.5
188
1233.0
189
1225.2
1089.5
1074.3
1093.0
1097.9



REFERENCE DRAWINGS

1. Legend — Dwg. File no. MGS 30I10
2. Plan of borings — Dwg. File no. MGS 30I8&9

REVISED TO SHOW "AS BUILT" CONDITIONS

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
RECORD OF BORINGS

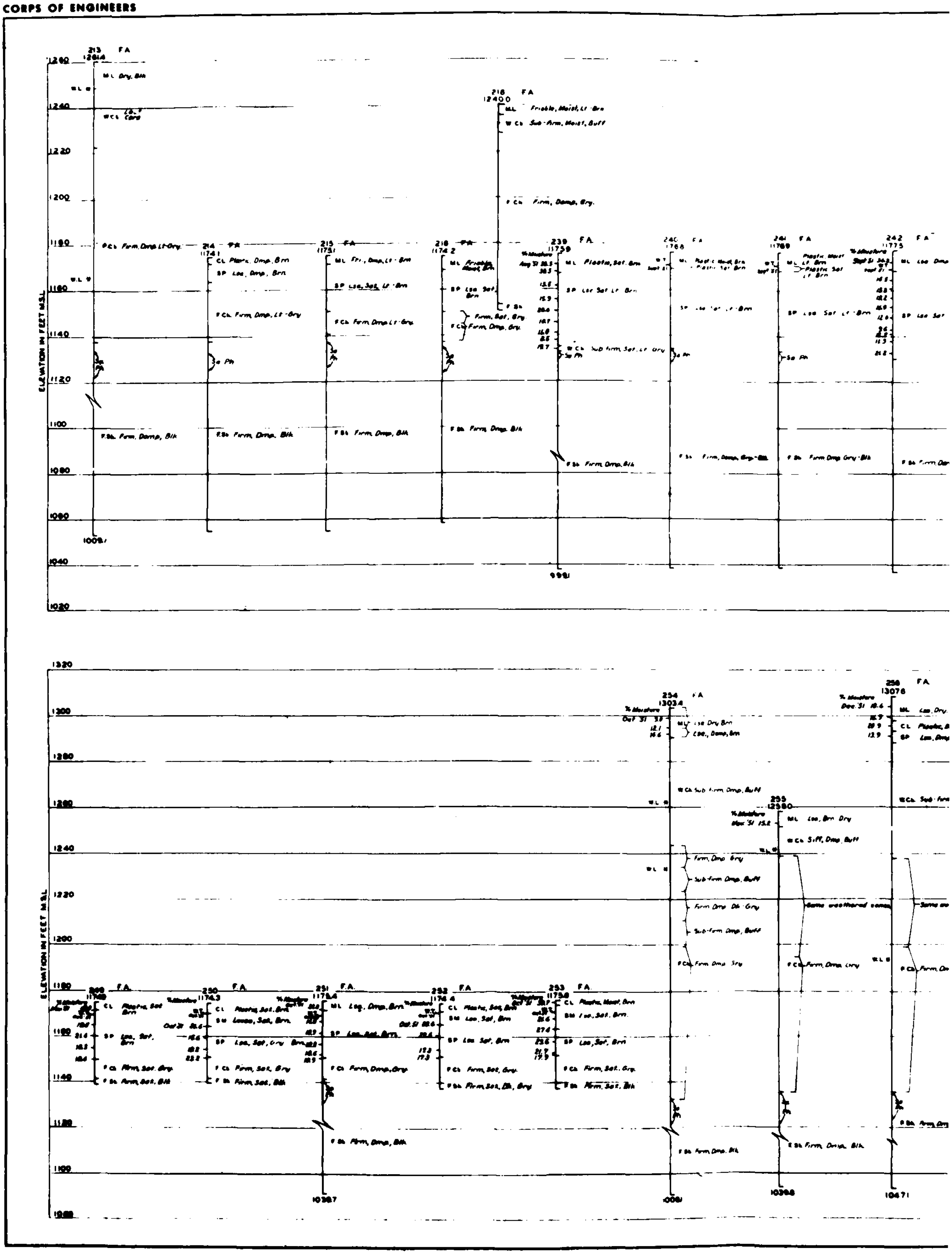
DRAWN BY A. E. R.
TRACED BY J. E. D.
CHECKED BY E. R. C.

COL. C. E. DISTRICT ENGINEER

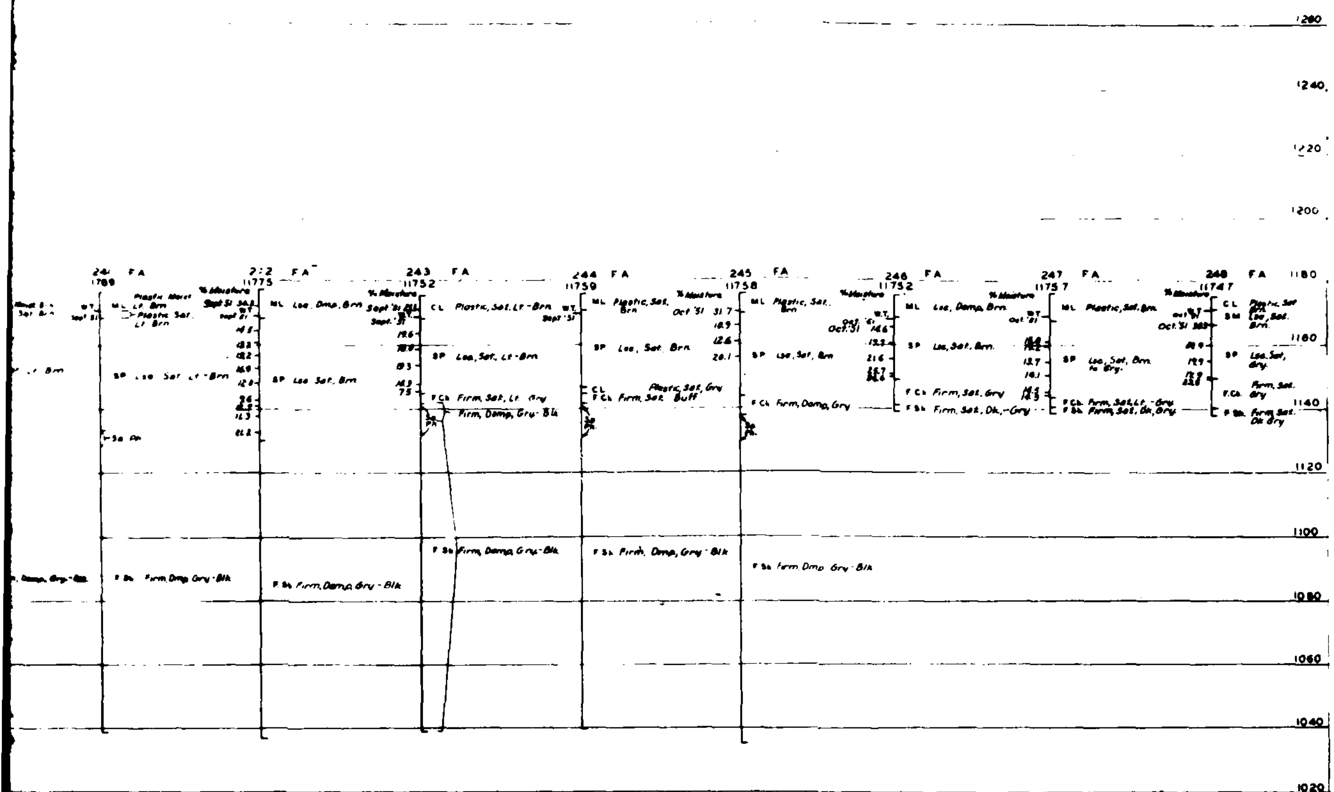
SCALE AS SHOWN

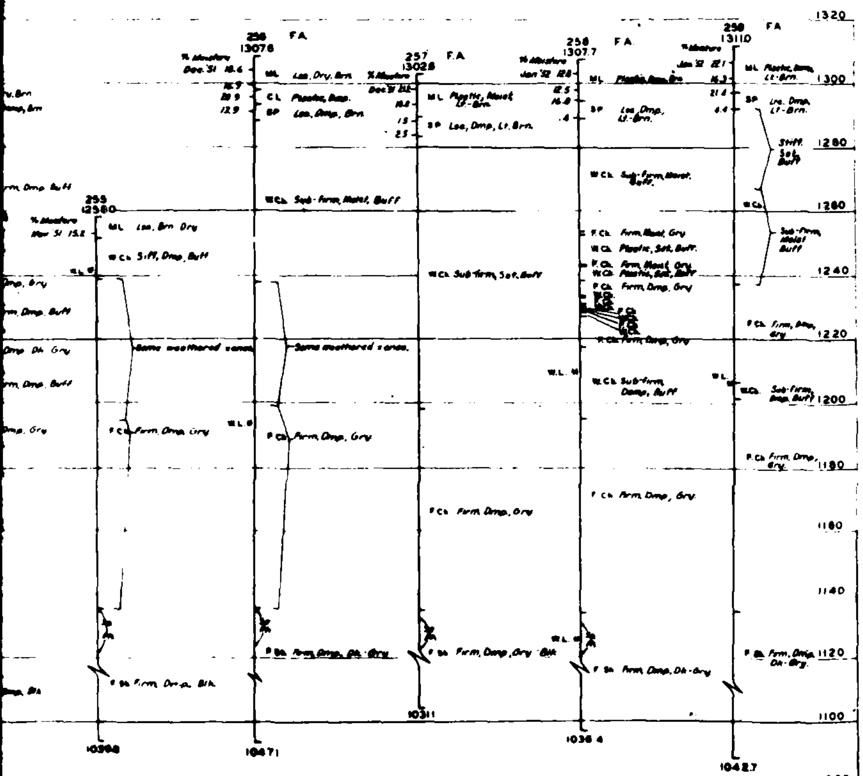
MGS-30E17.1

CORPS OF ENGINEERS



U. S. ARMY





REFERENCE DRAWINGS:

1. Legend — dwg. File no. MG5-30E10
2. Plan of borings — dwg File no MG5-30E8&9

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

1-16-57	REVISED TO SHOW "AS BUILT" CONDITIONS		
DATE	DESCRIPTION	MADE	APPR'D

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

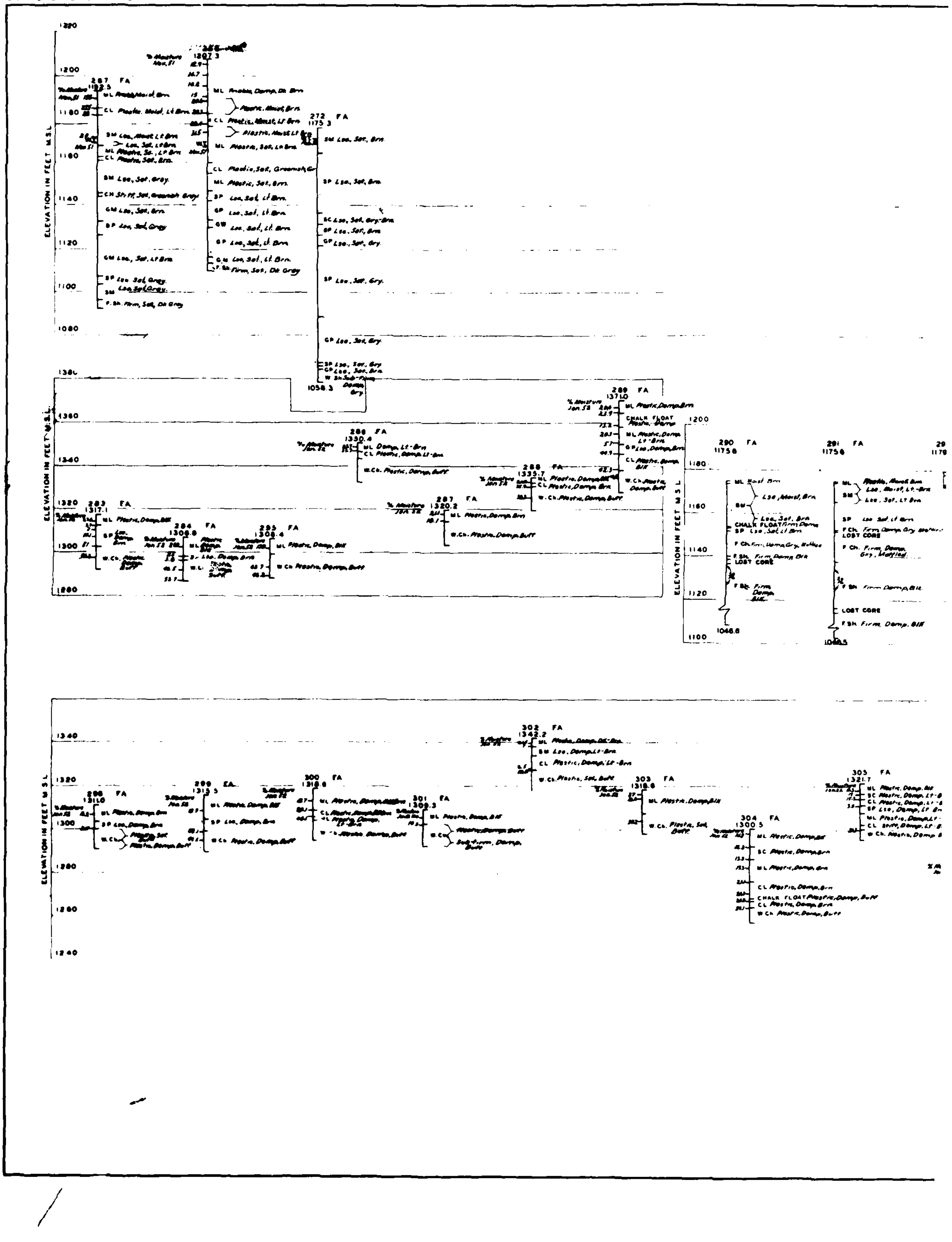
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
RECORD OF BORINGS

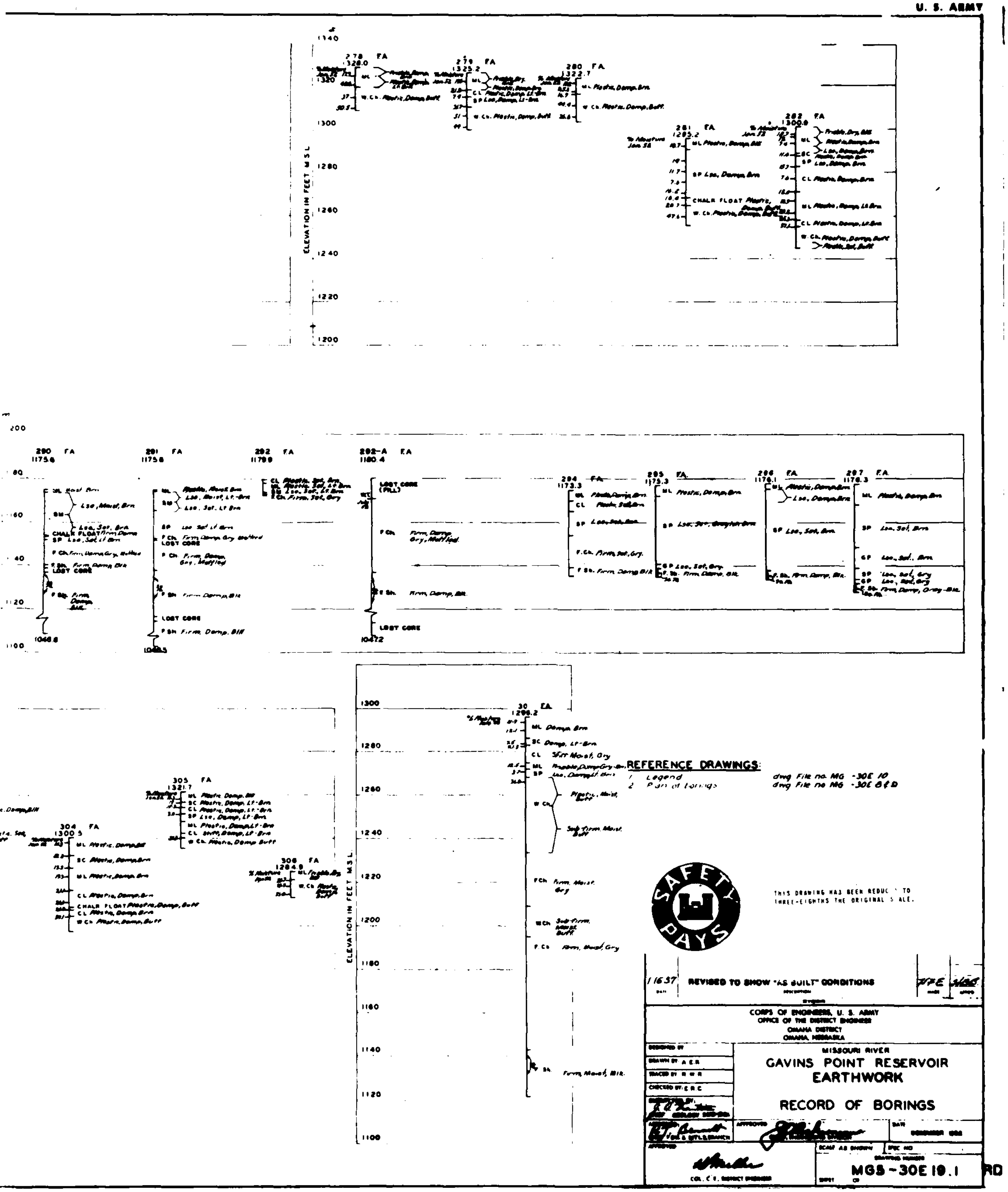
DESIGNED BY:
DRAWN BY: E.R.C.
TRACED BY: D.L.
CHECKED BY: A.E.R.

SCALE: AS SHOWN

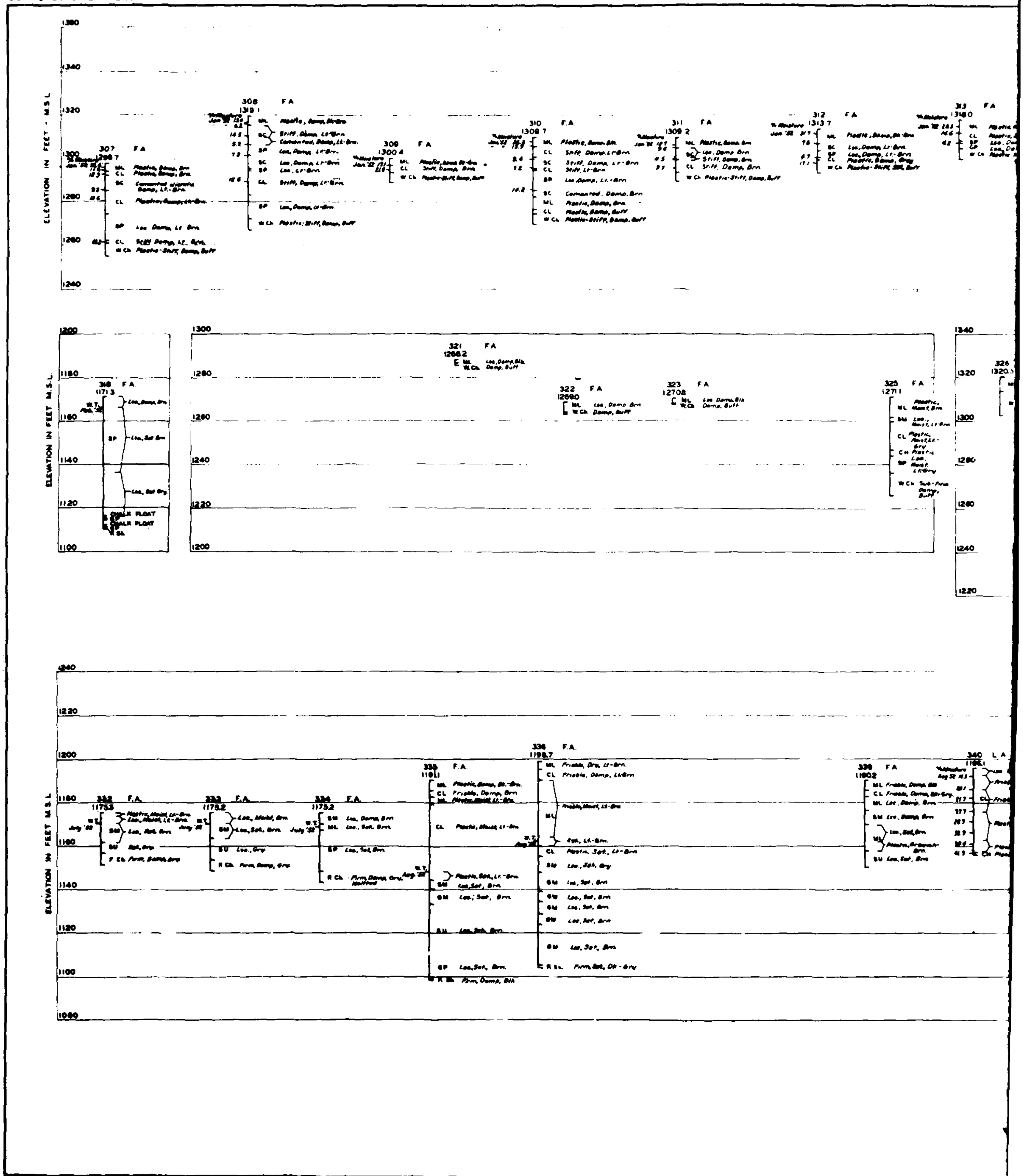
MG5-30E18.1

CORPS OF ENGINEERS

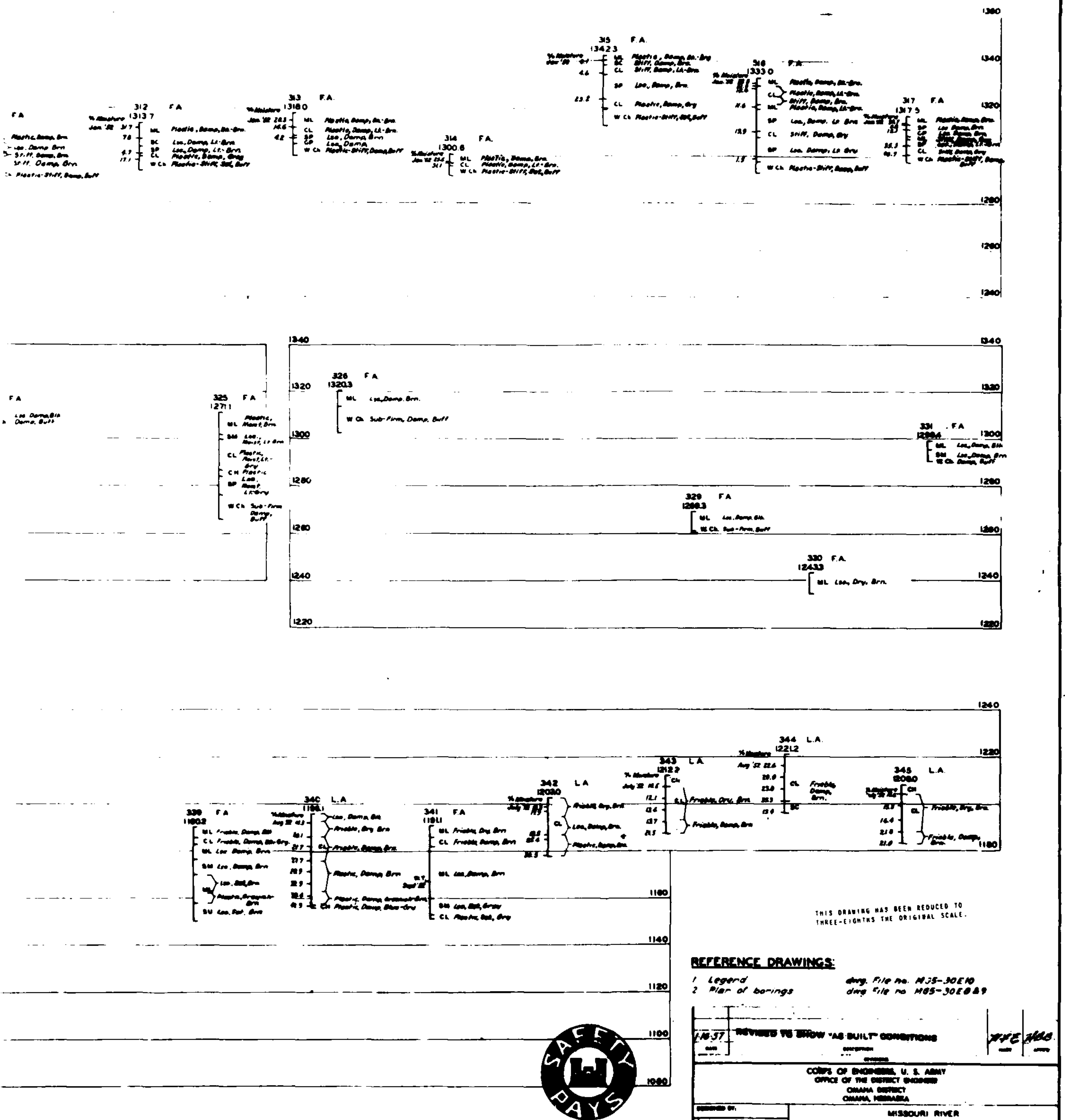



U. S. ARMY
ELEVATION IN FEET M.S.L.
REFERENCE DRAWINGS:
1 Legend dwg File no. MG -30E 10
2 Plan of borings dwg File no. MG -30E 8 & 9
THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.
SAFETY PAYS
REVISED TO SHOW "AS BUILT" CONDITIONS
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
RECORD OF BORINGS
MGS -30E 19.1

CORPS OF ENGINEERS



U. S. ARMY



THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

REFERENCE DRAWINGS:

1 Legend dwg. File no. MJS-30E10
2 Plan of borings dwg File no. MGS-30E8&9

1-16-57 REVISED TO SHOW "AS BUILT" CONDITIONS

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
RECORD OF BORINGS

DRAWN BY: E.R.G.
TRACED BY: D.W.H.
CHECKED BY: A.S.L.

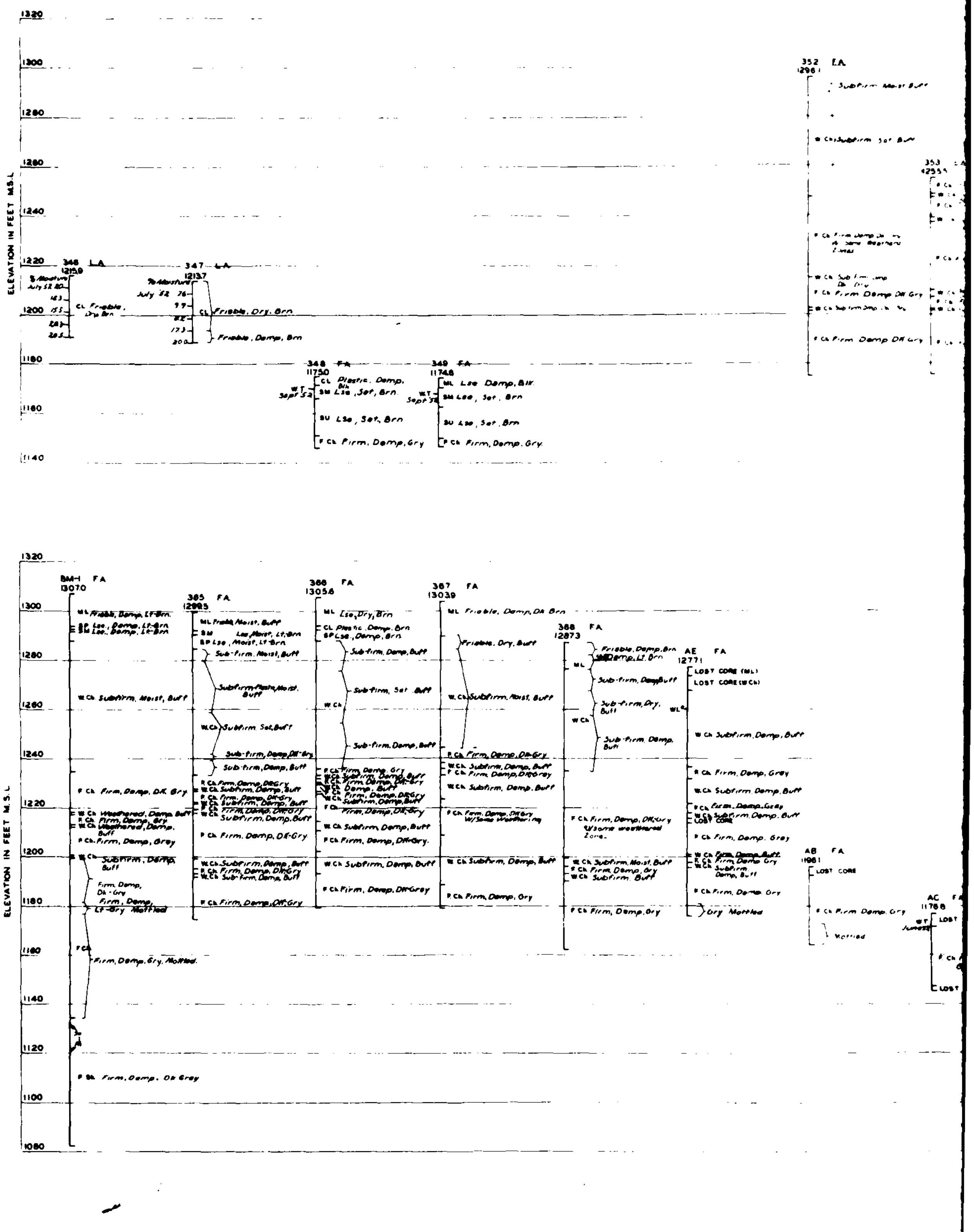
COL. C.E. DISTRICT ENGINEER

DECEMBER 1953

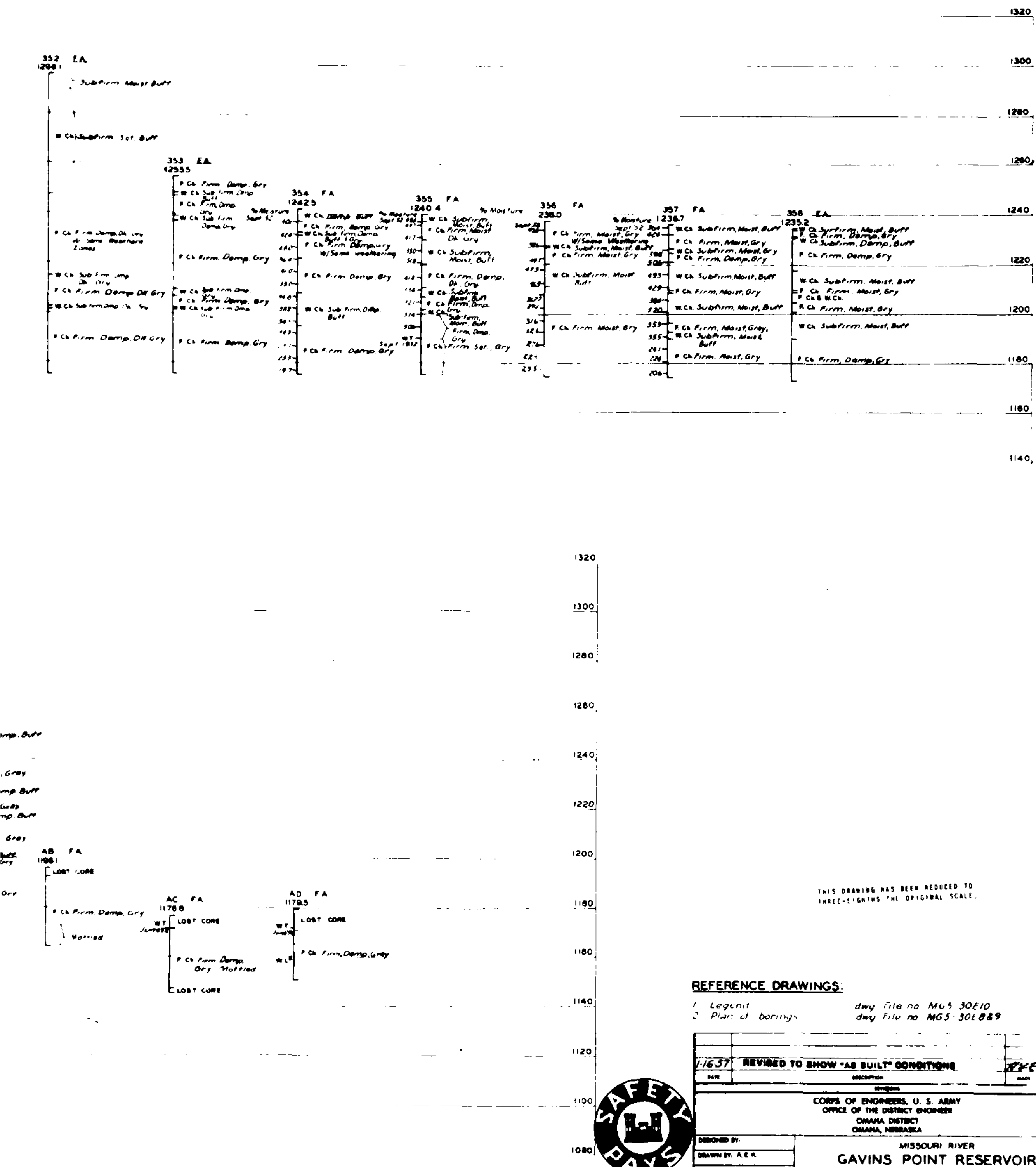
SCALE: AS SHOWN

MGS-30E20.1

CORPS OF ENGINEERS



U. S. ARMY



THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

REFERENCE DRAWINGS:

1. Legend — dwg File no MG5-30E10
2. Plan of borings — dwg File no MG5-30E889

DATE	DESCRIPTION	MADE	APPRD
1-16-57	REVISED TO SHOW "AS BUILT" CONDITIONS		

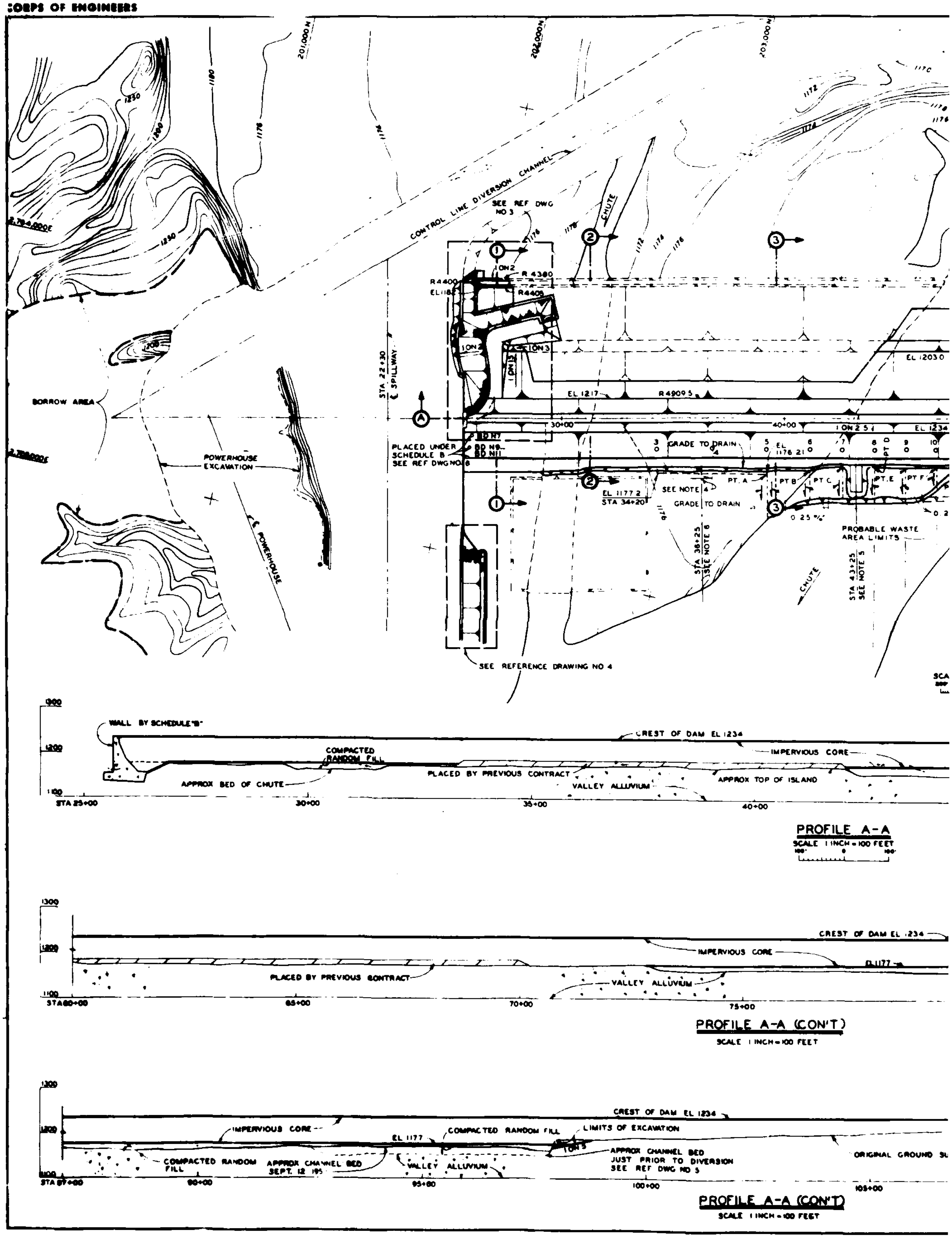
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

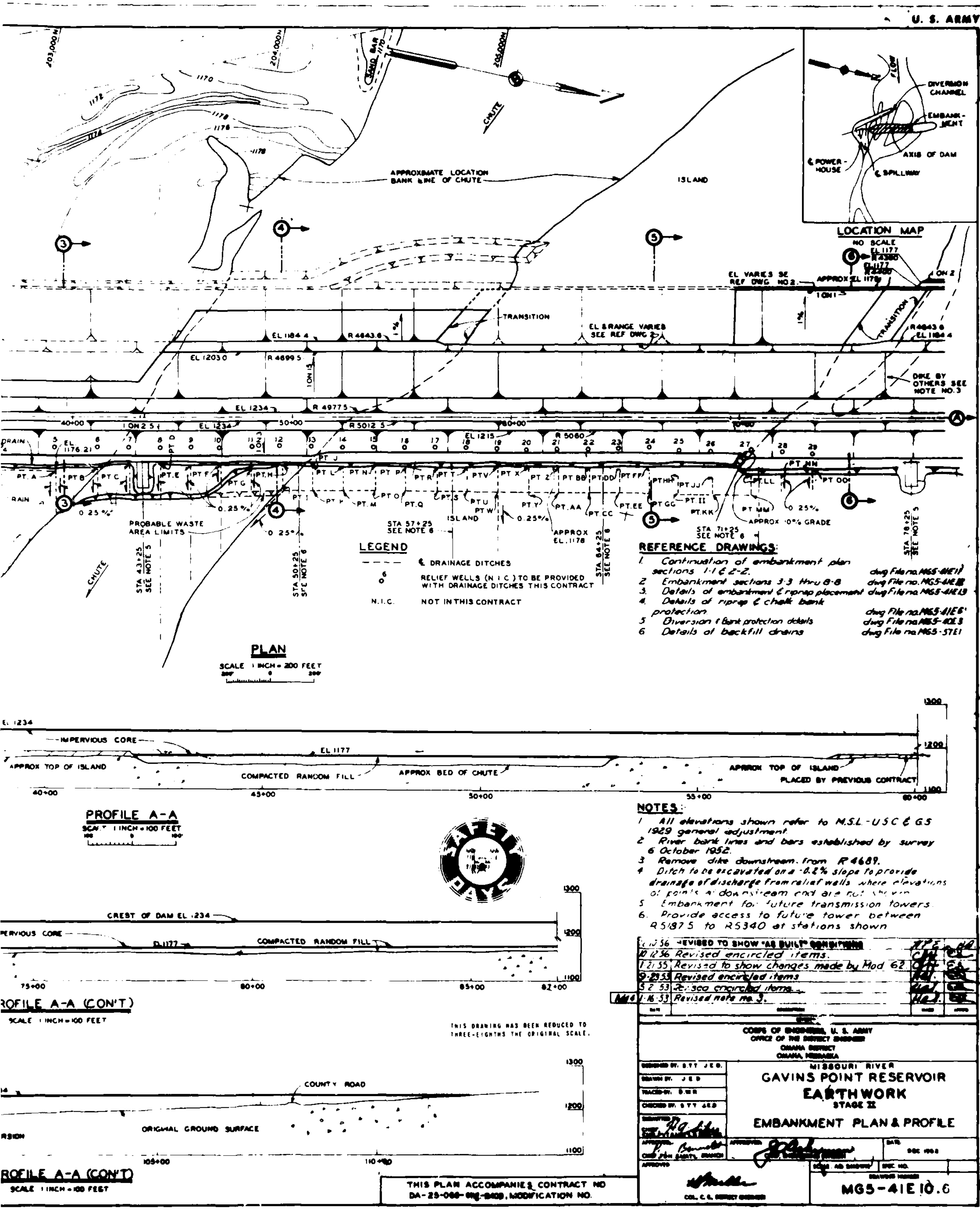
DESIGNED BY:
DRAWN BY: A.E.K.
TRACED BY: D.L.K.
CHECKED BY: E.R.C.
SUBMITTED BY: CHIEF, GEOLOGY SUB-SEC.
APPROVED: CHIEF, FDN. & MTLS. BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
APPROVED: COL. C.E., DISTRICT ENGINEER

MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
RECORD OF BORINGS

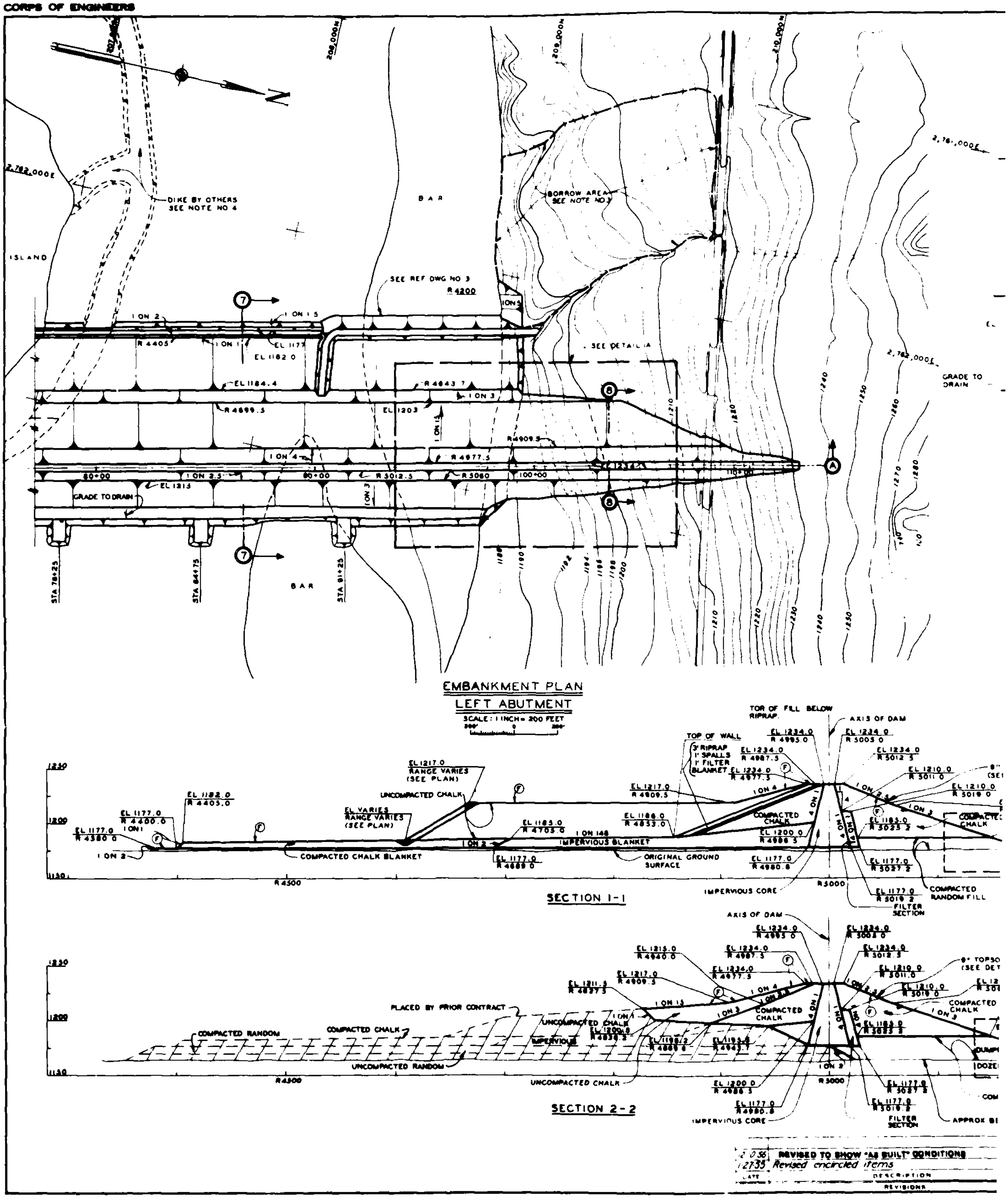
DATE: DECEMBER 1952
SCALE: AS SHOWN
DRAWING NUMBER: MG5-30E21.1

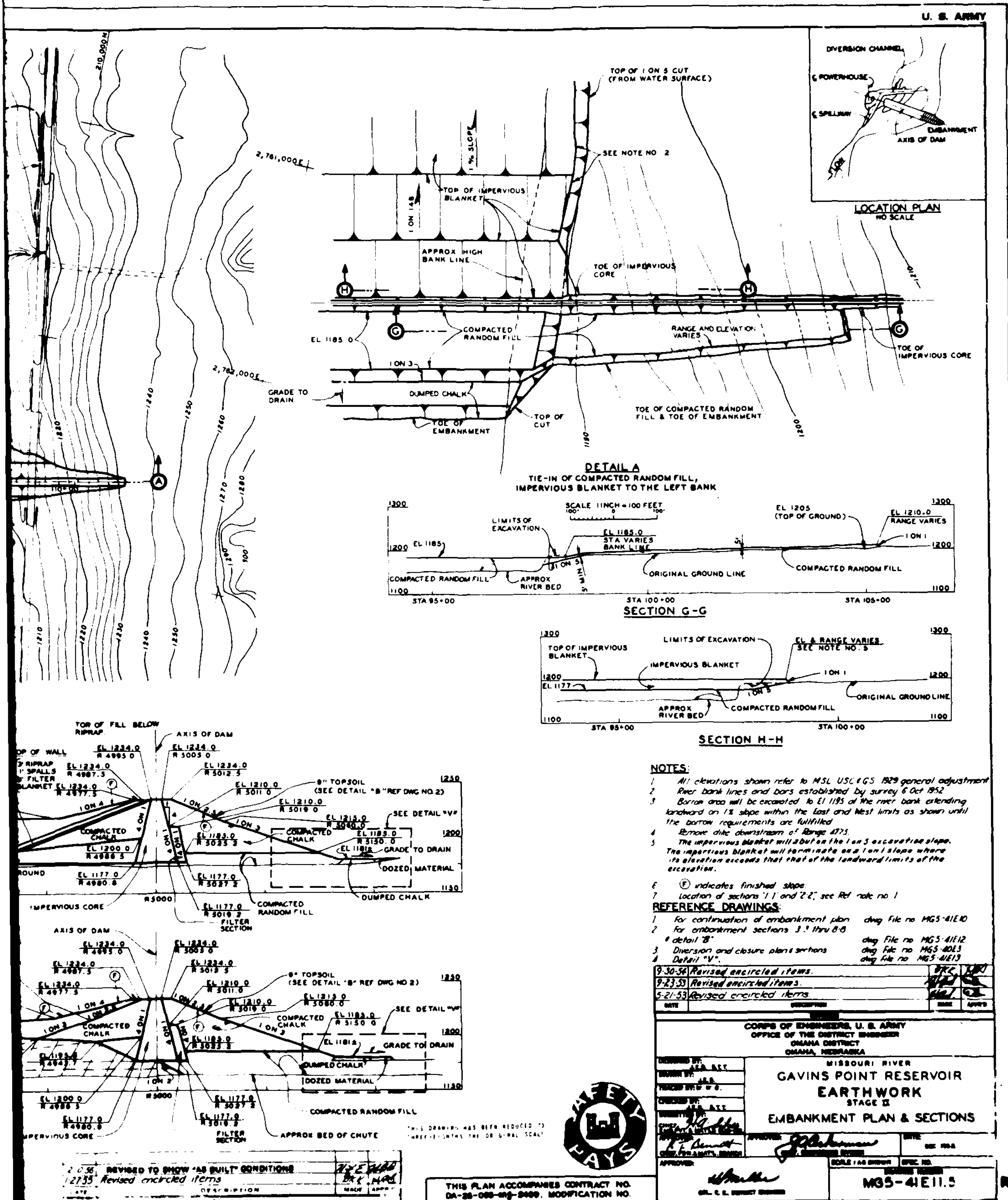
CORPS OF ENGINEERS




U. S. ARMY
LOCATION MAP
NO SCALE
DIVERSION CHANNEL
EMBANKMENT
AXIS OF DAM
℄ POWERHOUSE
℄ SPILLWAY
FLOW
APPROXIMATE LOCATION BANK LINE OF CHUTE
ISLAND
CHUTE
SAND BAR
TRANSITION
EL & RANGE VARIES SEE REF DWG 2
EL VARIES SE REF DWG NO 2
APPROX EL 1178
DIKE BY OTHERS SEE NOTE NO. 3
EL 1184.4
R 4643.6
EL 1203.0
R 4699.5
EL 1234
R 4977.5
R 5012.5
R 5060
EL 1215
PROBABLE WASTE AREA LIMITS
STA 43+25 SEE NOTE 5
STA 50+25 SEE NOTE 6
STA 57+25 SEE NOTE 6
STA 64+25 SEE NOTE 6
STA 71+25 SEE NOTE 6
STA 78+25 SEE NOTE 5
APPROX EL 1178
APPROX 0% GRADE
0.25%
LEGEND
℄ DRAINAGE DITCHES
RELIEF WELLS (N.I.C.) TO BE PROVIDED WITH DRAINAGE DITCHES THIS CONTRACT
N.I.C. NOT IN THIS CONTRACT
REFERENCE DRAWINGS:
1. Continuation of embankment plan sections 1-1 & 2-2. dwg File no. MG5-41E11
2. Embankment sections 3-3 thru 8-8 dwg File no. MG5-41E12
3. Details of embankment & riprap placement dwg File no. MG5-41E13
4. Details of riprap & chalk bank protection dwg File no. MG5-41E6
5. Diversion & Bank protection details dwg File no. MG5-40E3
6. Details of backfill drains dwg File no. MG5-57E1
PLAN
SCALE 1 INCH = 200 FEET
IMPERVIOUS CORE
EL 1177
APPROX TOP OF ISLAND
COMPACTED RANDOM FILL
APPROX BED OF CHUTE
PLACED BY PREVIOUS CONTRACT
PROFILE A-A
SCALE 1 INCH = 100 FEET
SAFETY DAYS
CREST OF DAM EL 1234
PROFILE A-A (CON'T)
SCALE 1 INCH = 100 FEET
COUNTY ROAD
ORIGINAL GROUND SURFACE
PROFILE A-A (CON'T)
SCALE 1 INCH = 100 FEET
NOTES:
1. All elevations shown refer to M.S.L.-U.S.C.& G.S. 1929 general adjustment.
2. River bank lines and bars established by survey 6 October 1952.
3. Remove dike downstream from R 4689.
4. Ditch to be excavated on a 0.2% slope to provide drainage of discharge from relief wells where elevations of points at downstream end are not shown.
5. Embankment for future transmission towers.
6. Provide access to future tower between R 5187.5 to R 5340 at stations shown.
REVISED TO SHOW "AS BUILT" CONDITIONS
10 12 56 Revised encircled items.
1 21 55 Revised to show changes made by Mod 62
9-29-53 Revised encircled items
5 2 53 Revised encircled items.
1-16-53 Revised note no. 3.
THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
EMBANKMENT PLAN & PROFILE
THIS PLAN ACCOMPANIES CONTRACT NO DA-25-066-ENG-2409, MODIFICATION NO.
MG5-41E10.6

CORPS OF ENGINEERS





NOTES:

1. All elevations shown refer to MSL USC&GS 1929 general adjustment
2. River bank lines and bars established by survey 6 Oct 1952
3. Borrow area will be excavated to El 1195 at the river bank extending landward on 1% slope within the East and West limits as shown until the borrow requirements are fulfilled
4. Remove dike downstream of Range 4775.
5. The impervious blanket will abut on the 1 on 5 excavation slope. The impervious blanket will terminate on a 1 on 1 slope where its elevation exceeds that that of the landward limits of the excavation.
6. (F) indicates finished slope
7. Location of sections "1-1" and "2-2", see Ref note no 1

REFERENCE DRAWINGS:

1. For continuation of embankment plan — dwg File no MGS-41E10
2. For embankment sections 3-3 thru 8-8 & detail "B" — dwg File no MGS-41E12
3. Diversion and closure plans & sections — dwg File no MGS-40E3
4. Detail "V". — dwg File no MGS-41E13

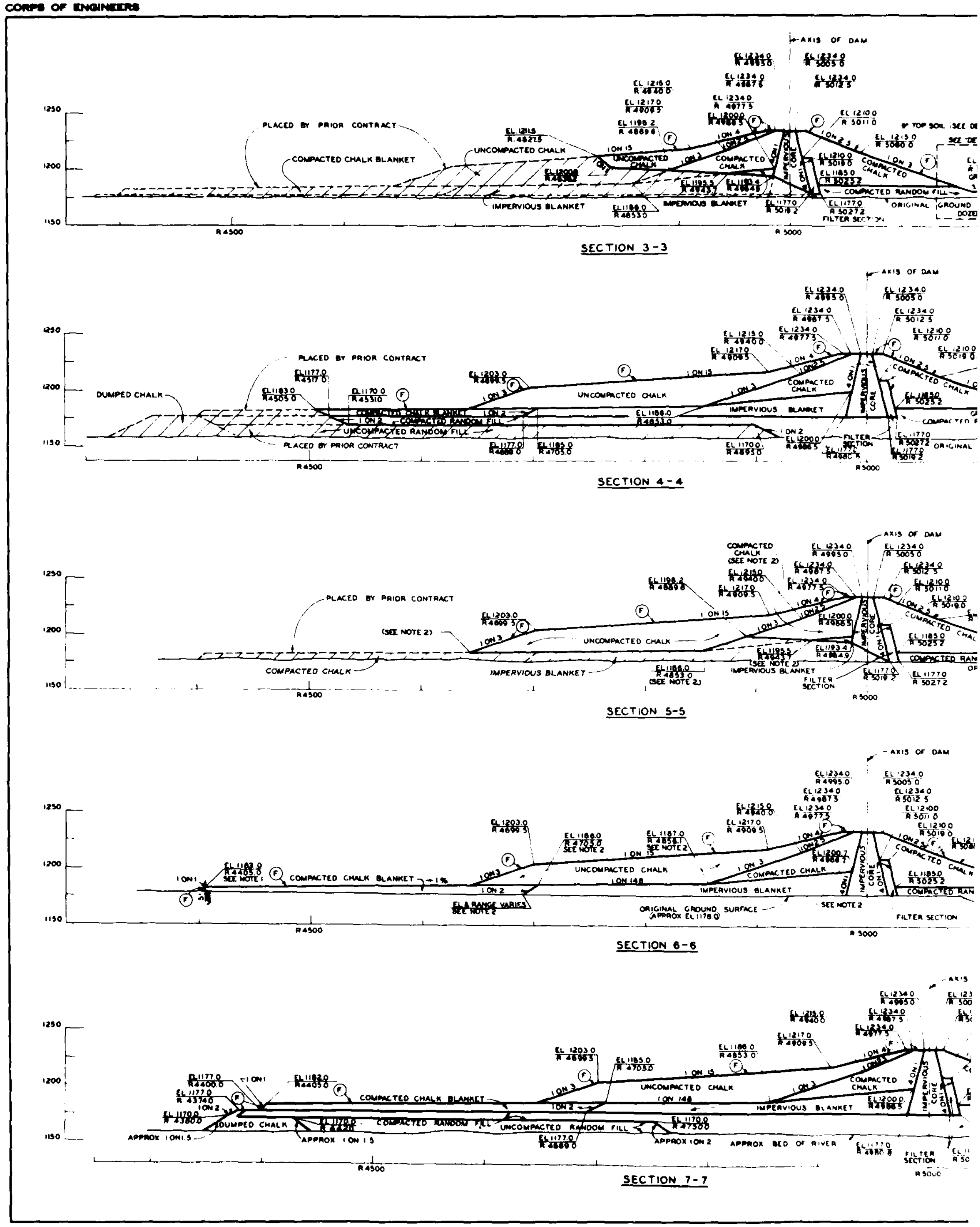
DATE	DESCRIPTION	MADE	APPR'D
9-30-54	Revised encircled items.		
9-23-53	Revised encircled items.		
5-21-53	Revised encircled items.		

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

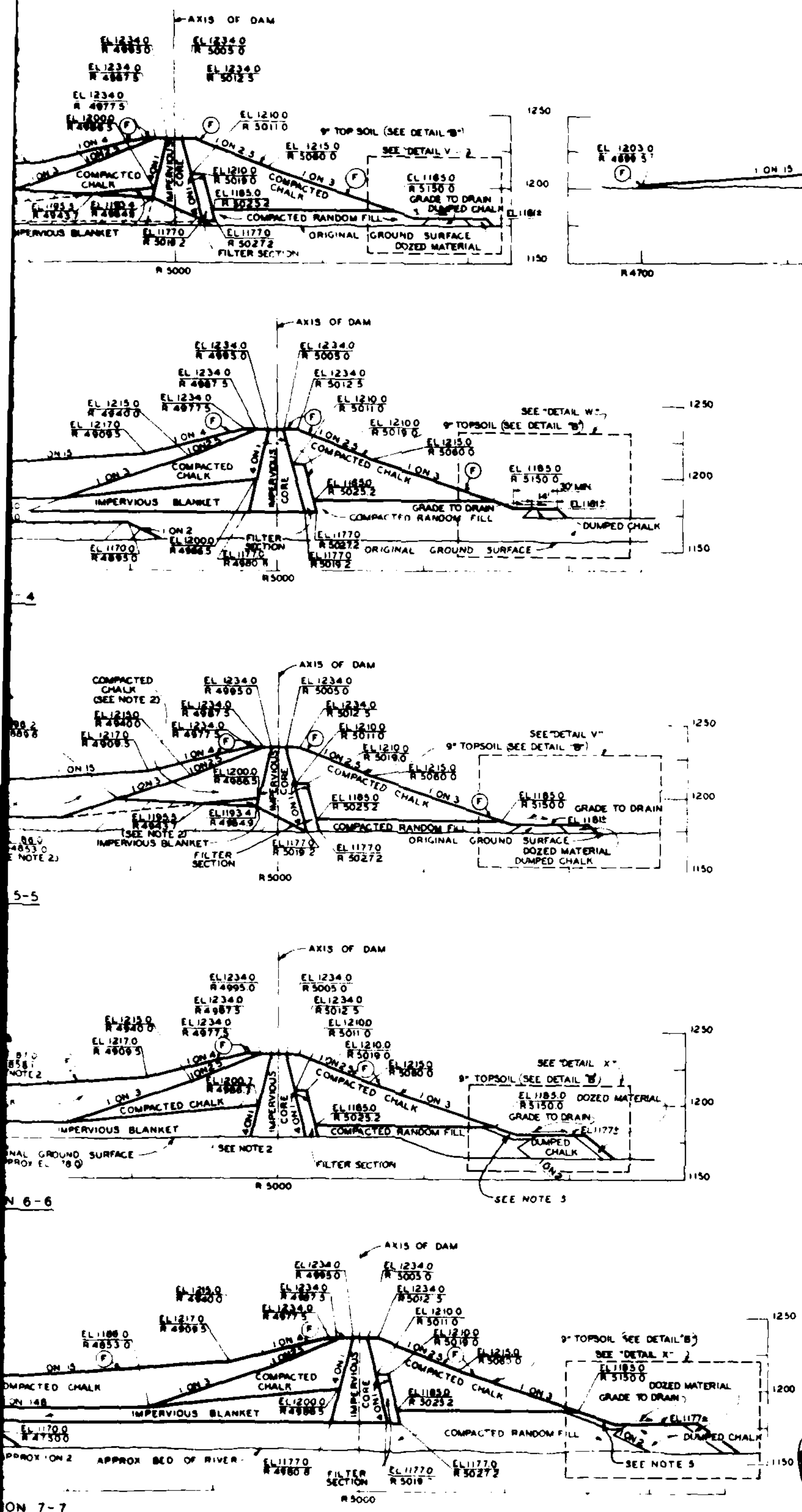
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
EMBANKMENT PLAN & SECTIONS

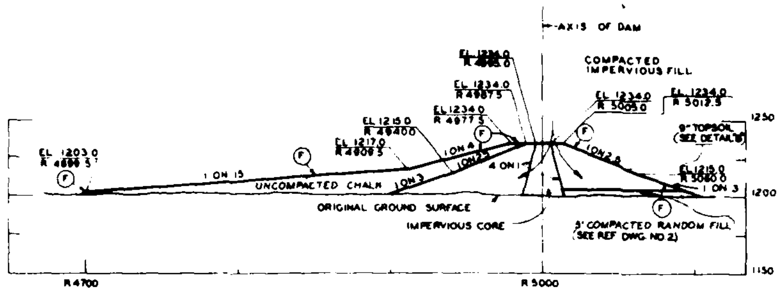
MGS-41E11.5

CORPS OF ENGINEERS

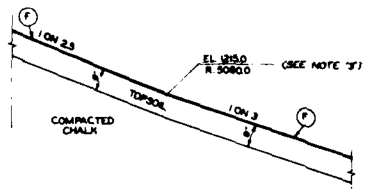


U. S. ARMY





SECTION 8-8



DETAIL "B"
SCALE 1"=2'

REFERENCE DRAWINGS — DWG. NO.

1	For locations of sections 3-3 thru 6-6	MG5-41E10
2	For locations of sections 7-7 & 8-8 and detailed treatment of compacted random fill	MG5-41E16
3.	Details "V", "W" and "X".	MG5-41E13

NOTES:

1. All elevations refer to M.S.L. U.S.C.& G.S. 1929 general adjustment
2. In the event that the existing ground line varies from that shown on the island, a minimum thickness of five (5) feet shall be maintained at the upstream end of the compacted chalk blanket, the grades on the top of the compacted chalk, and impervious blankets will be maintained and the range ℄ elevations of the breaks in grade will vary accordingly. In areas where the compacted chalk blanket ℄ portions of the impervious blanket have been placed under prior contract, the ranges ℄ elevations of breaks in grade on work under this contract may vary accordingly.
3. The finished surface of the embankment including a 9" layer of topsoil is defined on the embankment sections
4. (F) indicates finished slopes
5. 2' layer dumped chalk, extending from Sta 70+50, North to the river channel.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D
12-10-56	REVISED TO SHOW "AS BUILT" CONDITIONS		
9-29-54	Revised encircled items		
9-23-53	Revised encircled items		
5-21-53	Revised encircled items		

REVISIONS

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

DESIGNED BY: J.E.D. S.T.T.
DRAWN BY: J.E.D.
TRACED BY: D.W.R.
CHECKED BY: J.E.D. S.T.T.

MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
EMBANKMENT SECTIONS

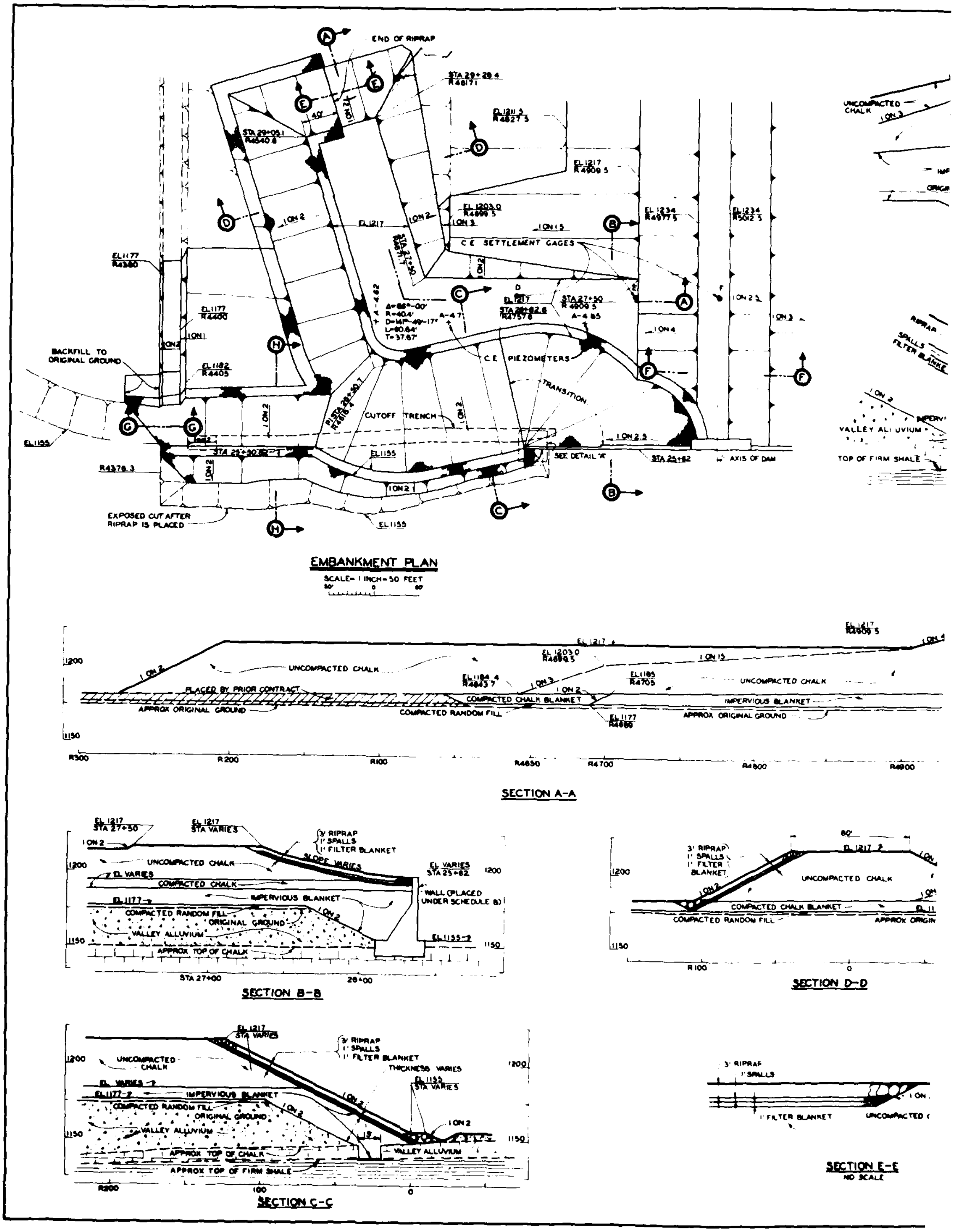
SUBMITTED BY: CHIEF, EMB., FDN & MATL'S SUB-SEC.
APPROVED: CHIEF, FDN & MAT'L BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
DATE: MAY 1953
SCALE: AS SHOWN
SPEC NO.
APPROVED: COL. C. E. DISTRICT ENGINEER
DRAWING NUMBER: MG5-41E 12.4

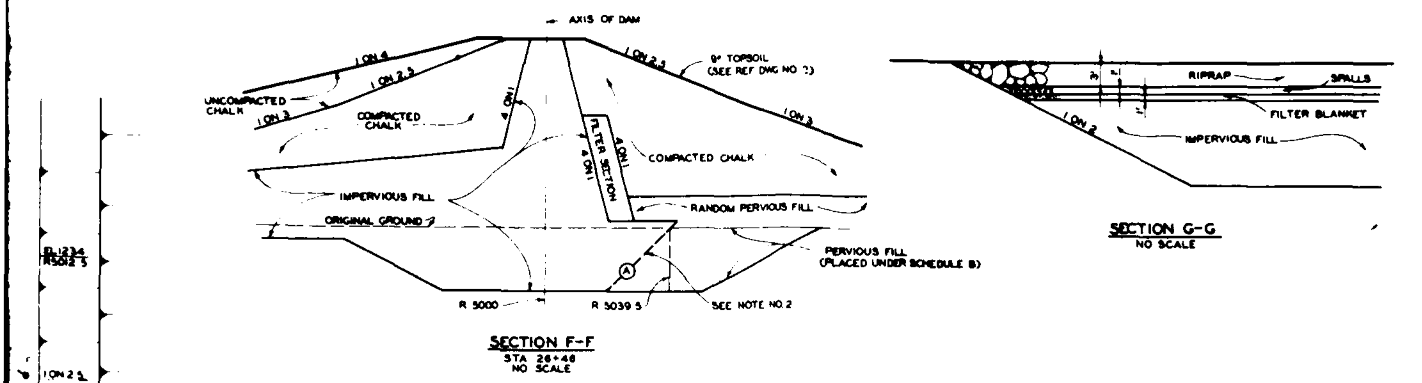
SAFETY PAYS

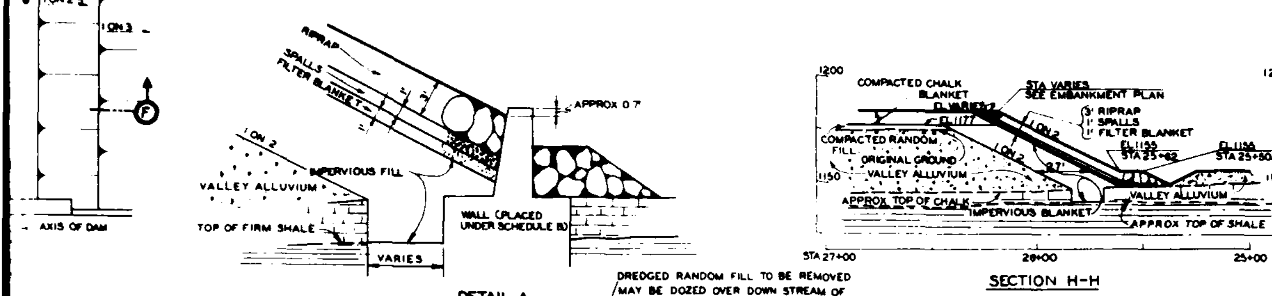
THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-066-ENG-2409, MODIFICATION NO.

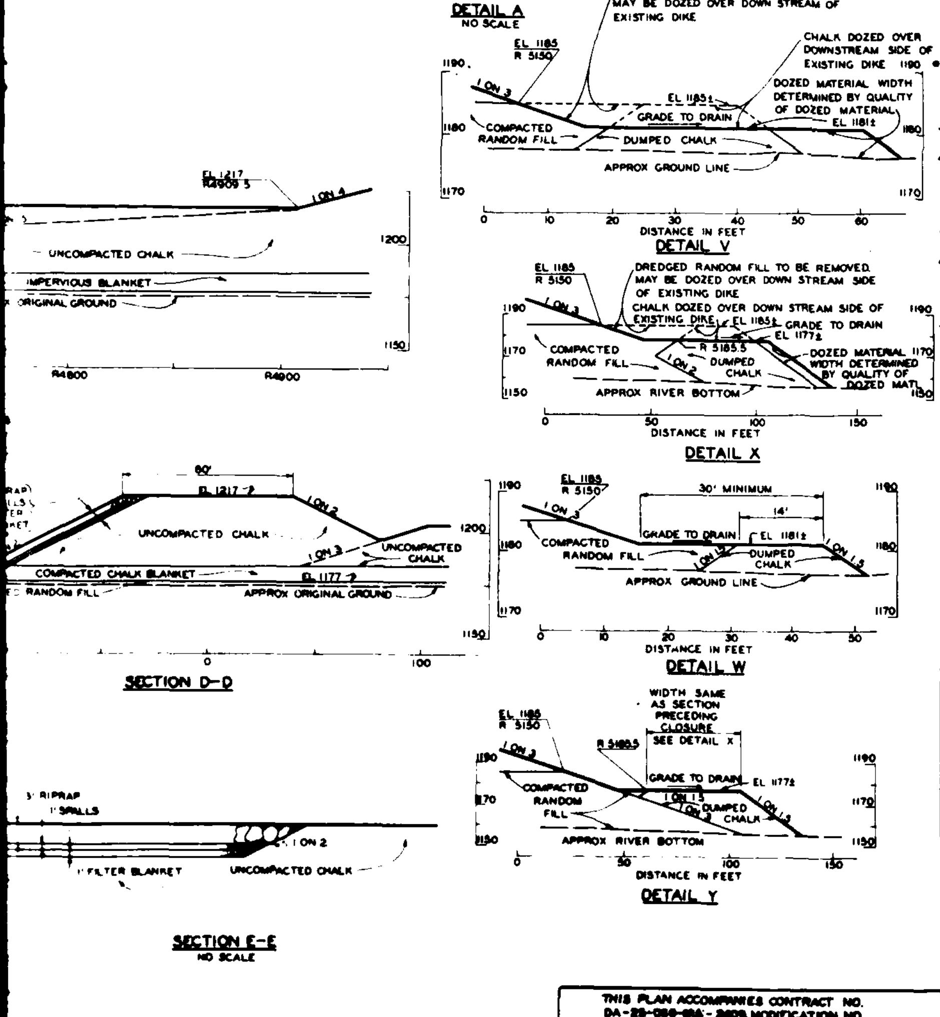
2

CORPS OF ENGINEERS









REFERENCE DRAWINGS:

1. Excavation details — dwg file no MG5-41E4 & 41E5
2. Topsoil details — dwg file no MG5-41E12

NOTES:

1. All elevations shown refer to M.S.L.-U.S.C. & G.S. 1929 general adjustments
2. The boundary line between the impervious backfill placed under this contract and the pervious backfill placed by Schedule B, will be line "A" or line "B" depending upon which type of backfill is placed first.
3. Between Range 4759.5 & Range 4789.5 the riprap, spalls & filter are not keyed into the toe of the slope. See "Detail A"
4. Details V, W, X & Y indicate the final treatment of the chalk protection at the downstream toe of the embankment incorporating the excess dumped chalk and compacted random fill placed by the contractor for his convenience in connection with the placement of the compacted random fill by dredge method.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION
12-10-56	REVISED TO SHOW "AS BUILT" CONDITIONS
11-4-53	Revised encircled items.
9-25-53	Revised encircled items
5-21-53	Revised and Redrawn

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

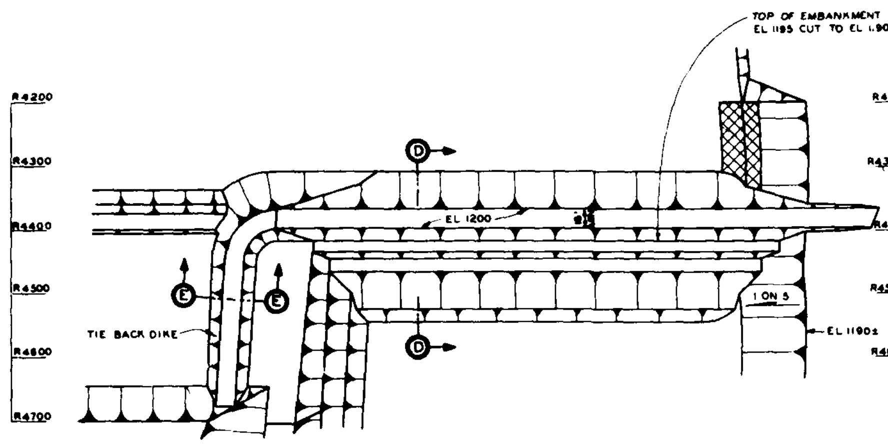
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
EMBANKMENT DETAILS

DATE: MAY 1953

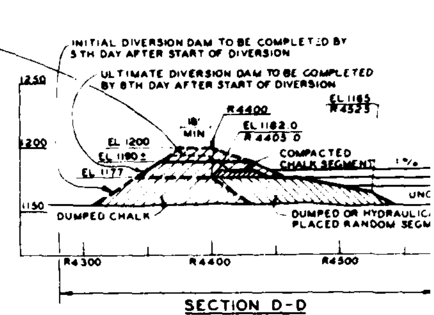
MG5-41E13.4

THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-066-eng-2409 MODIFICATION NO.

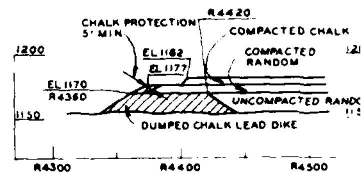
CORPS OF ENGINEERS



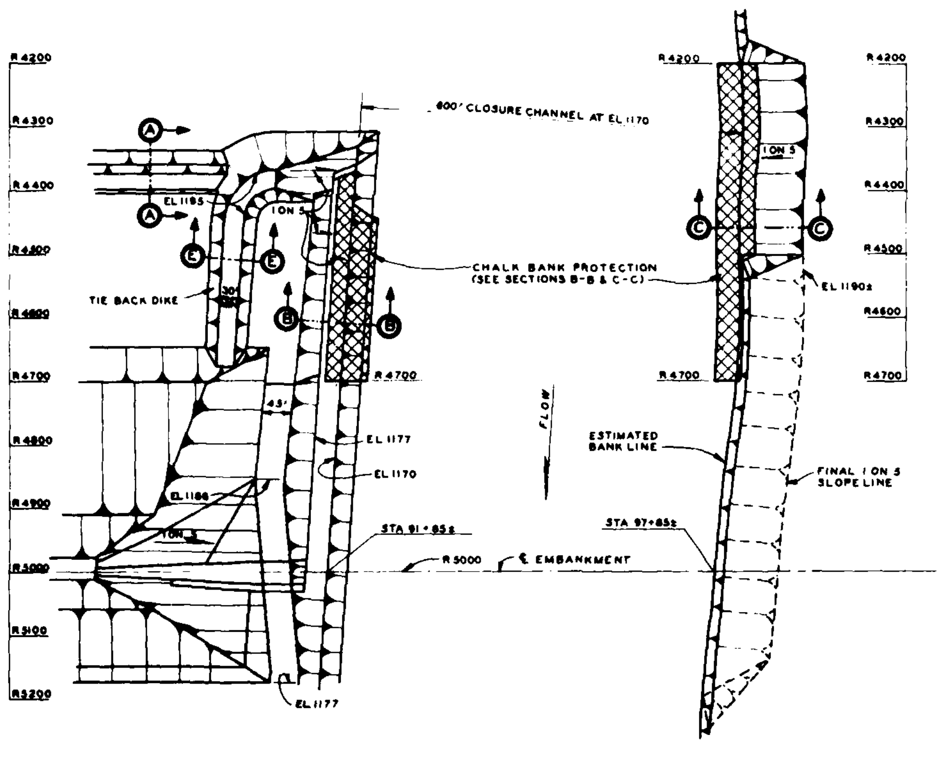
PLAN
COMPLETED DIVERSION DAM
SCALE 1 INCH=100 FEET



SECTION D-D
SECTIONS OF DIVERSION DAM



SECTION A-A
LEAD DIKE

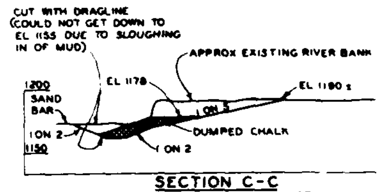


PLAN
CONDITION OF CHANNEL BEFORE DIVERSION
SCALE 1 INCH=100 FEET

SECTION B-B
SCALE 1 INCH=50 FEET VERT HOR

CHALK PROTECTION ON RIGHT CHANNEL BANK

SHALL BE PLACED TO LIMITS SHOWN ON PLAN AS SOON AS RIGHT BANK SECTION OF EMBANKMENT HAS REACHED 600' CHANNEL LINE AND SHALL BE REMOVED BEFORE ADDITIONAL EMBANKMENT IS PLACED IN THIS AREA.

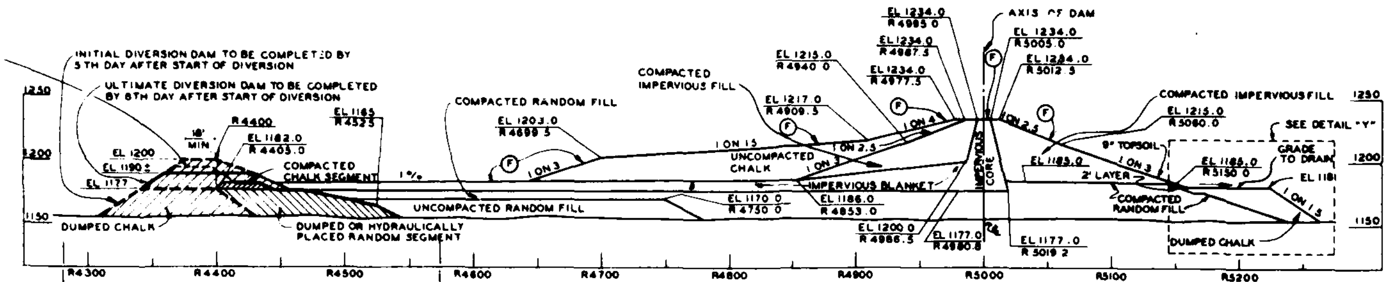


SECTION C-C
SCALE 1 INCH=50 FEET VERT HOR

CHALK PROTECTION ON LEFT RIVER BANK

SHALL BE PLACED FROM R4200 TO R4700 ON EITHER EXISTING OR SLOPED RIVER BANK BEFORE 15 APRIL 1953.
WHERE RANDOM FILL IS TO BE PLACED, CHALK PROTECTION SHALL BE REMOVED BEFORE PLACEMENT OF RANDOM.
RIVER BANK SHALL BE CUT BACK TO 1 ON 5 SLOPE FROM WATER EDGE FROM R4200 TO R4500 BEFORE DIVERSION.

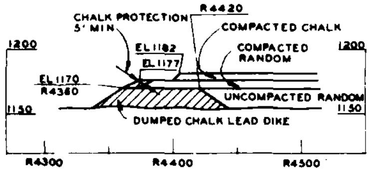
SECTION E-E
SCALE 1 INCH=50 FEET VERT HOR



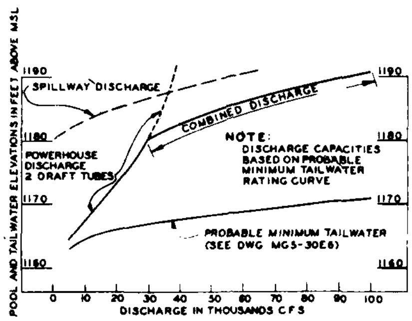
Ⓕ DENOTES FINISH SLOPE

SECTION D-D
SECTIONS OF DIVERSION DAM

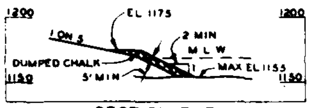
TYPICAL SECTION
CLOSURE EMBANKMENT



SECTION A-A
LEAD DIKE



DIVERSION RATING CURVE
USING 2 DRAFT TUBES AND SPILLWAY

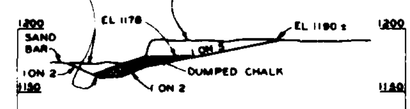


SECTION B-B
SCALE 1 INCH = 50 FEET VERT HOR

CHALK PROTECTION ON RIGHT CHANNEL BANK
SHALL BE PLACED TO LIMITS SHOWN ON PLAN AS SOON AS RIGHT BANK SECTION OF EMBANKMENT HAS REACHED 600' CHANNEL LINE AND SHALL BE REMOVED BEFORE ADDITIONAL EMBANKMENT IS PLACED IN THIS AREA.

REFERENCE DRAWINGS:

1. *Continuation of Embankment Plan* — *dwg File no. MG5-41E11*
2. *Continuation of Embankment Sections* — *dwg File no. MG5-41E12*
3. *Detail "Y"* — *dwg File no. MG5-41E13*



SECTION C-C
SCALE 1 INCH = 50 FEET VERT HOR

CHALK PROTECTION ON LEFT RIVER BANK
SHALL BE PLACED FROM R4200 TO R4700 ON EITHER THE EXISTING OR SLOPED RIVER BANK BEFORE 15 APRIL 1953.
WHERE RANDOM FILL IS TO BE PLACED, CHALK PROTECTION SHALL BE REMOVED BEFORE PLACEMENT OF RANDOM.
RIVER BANK SHALL BE CUT BACK TO 1 ON 5 SLOPE FROM WATER'S EDGE FROM R4800 TO R4500 BEFORE DIVERSION.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

12-10-56	REVISED TO SHOW "AS BUILT" CONDITIONS			
9-29-54	*Revised encircled items.*			
9-29-53	*Revised encircled items.*			
5-21-53	*Revised Typical Section - Closure Embankment*			
Add 3 1-9-53	*Revised date in note under "Section C-C".*			
DATE	DESCRIPTION		MADE	APPR'D

SECTION E-E
SCALE: 1 INCH = 50 FEET VERT HOR

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

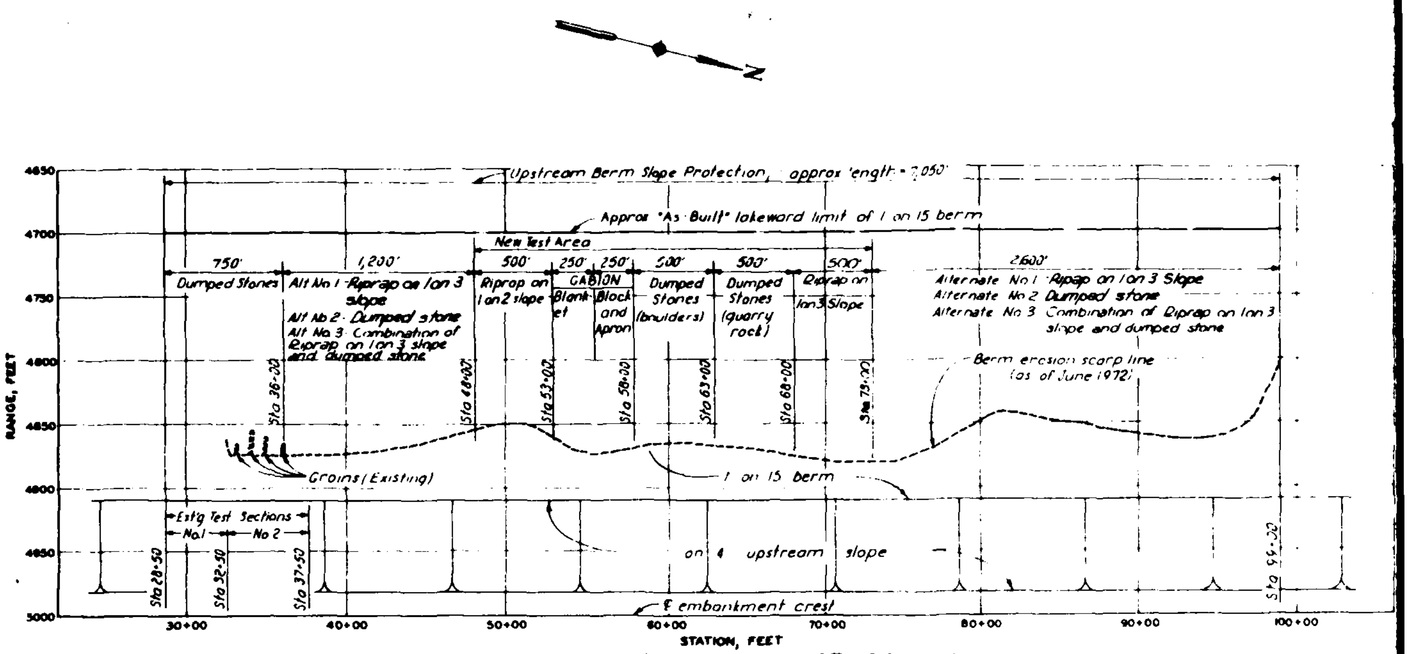
MISSOURI RIVER
GAVINS POINT RESERVOIR
EARTHWORK
STAGE II
DIVERSION AND CLOSURE
PLANS AND SECTIONS

DESIGNED BY:
DRAWN BY: H.W.A.
TRACED BY: W.W.S.
CHECKED BY: J.N.G.
SUBMITTED BY:
APPROVED:
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
MG5-40E3.5

COL., C. E. DISTRICT ENGINEER

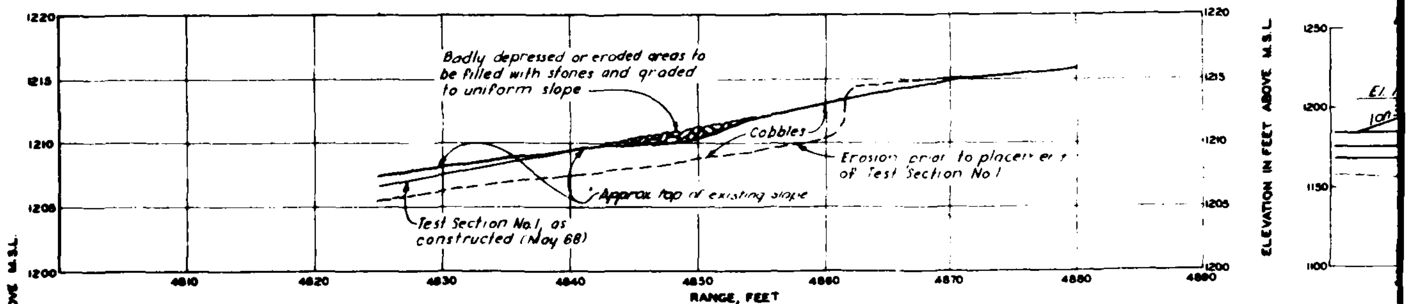
THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-066-eng.-2409, MODIFICATION NO.

2



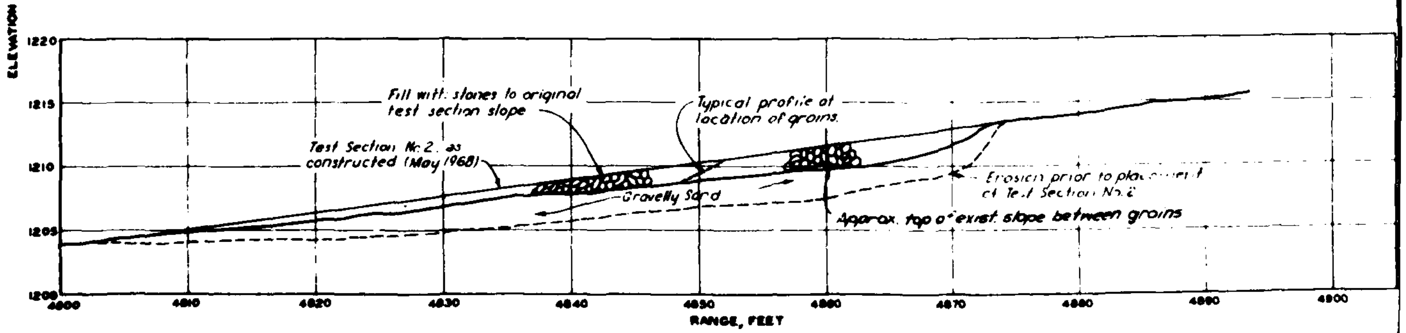
LIMITS OF UPSTREAM BERM PROTECTION PLAN

SCALE VERT 1 INCH = 50 FEET
HORIZ 1 INCH = 400 FEET



SLOPE PROTECTION - STA. 28+50 TO STA. 32+50

SCALE 1 INCH = 5 FEET

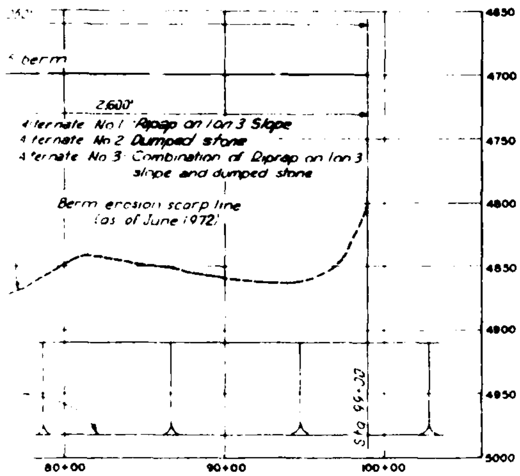


SLOPE PROTECTION - STA. 32+50 TO STA. 36+00

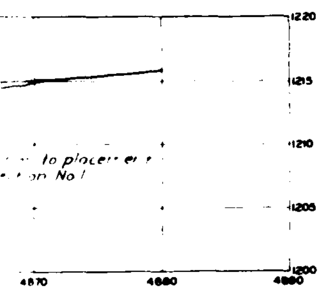
SCALE 1 INCH = 5 FEET

5' 0 5'

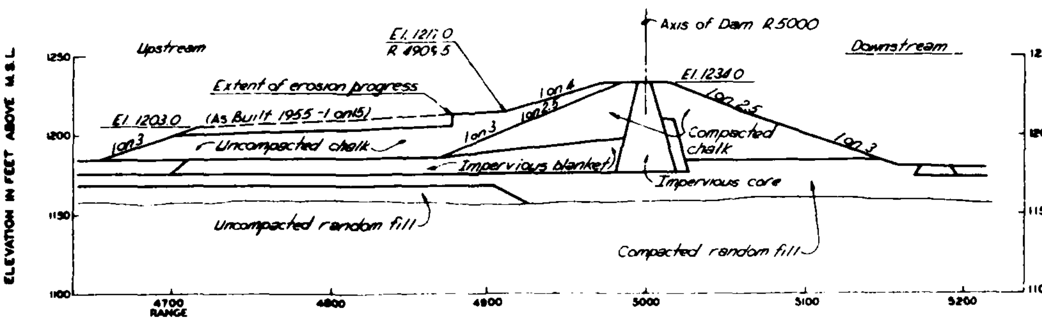
EMB



PLAN

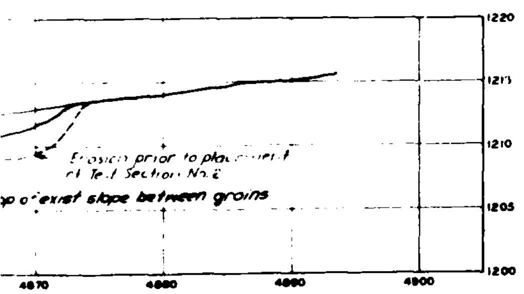


A 32+50



TYPICAL EMBANKMENT SECTION

SCALE 1 INCH = 40 FEET



TA. 36+00

NOTES:

1. Sta. 28+50 to Sta. 36+00. Dumped stones over existing test section, as shown on Typical Sections, this sheet.
2. Sta. 36+00 to Sta. 48+00 (Alternate No. 1 or Alternate No. 2).
3. Sta. 48+00 to Sta. 73+00 New Test Area.
4. Sta. 73+00 to Sta. 99+00 (Alternate No. 1 or Alternate No. 2).

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

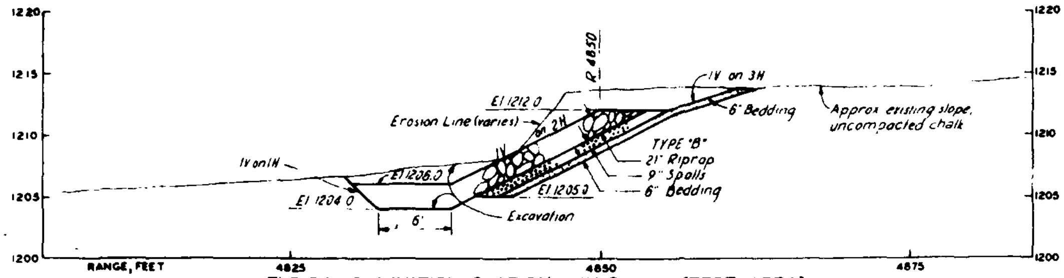
MISSOURI RIVER
GAVINS POINT DAM-LEWIS AND CLARK LAKE
UPSTREAM BERM SLOPE PROTECTION
METHODS OF SLOPE PROTECTION
PLAN AND TYPICAL SECTIONS

DATE: DEC 1972

SAFETY PAYS

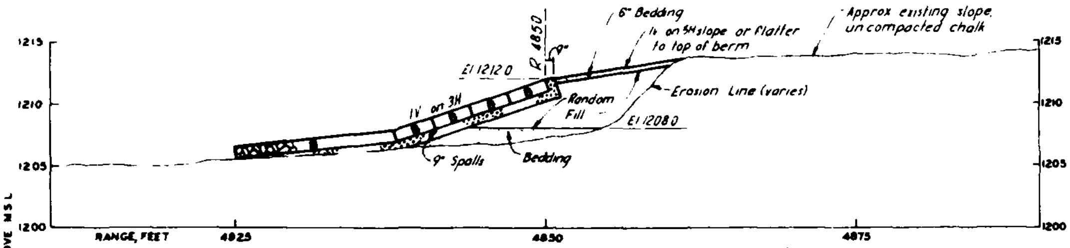
THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

MG174-40E3



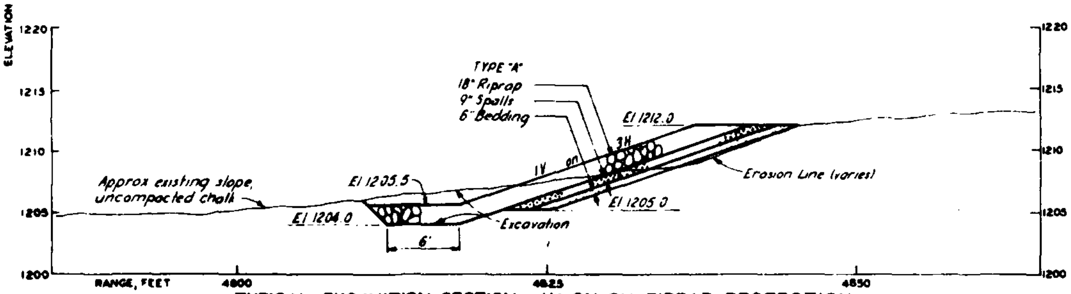
TYPICAL EXCAVATION SECTION - IV ON 2H (TEST AREA)

STA. 48+00 THRU STA. 53+00



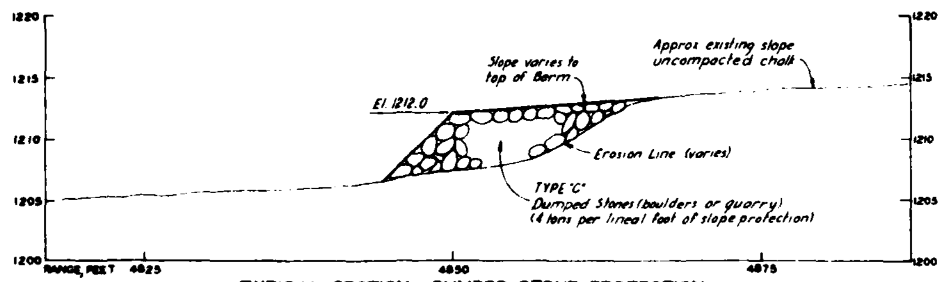
TYPICAL SECTION - GABION PROTECTION (TEST AREA)

BLANKET TYPE - STA. 53+00 THRU 56+50



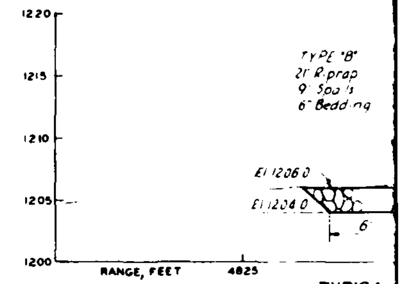
TYPICAL EXCAVATION SECTION - IV ON 3H RIPRAP PROTECTION

ALTERNATE NO. 1: STA. 36+00 THRU STA. 48+00 & STA. 73+00 THRU STA. [illegible]
STA. 68+00 THRU STA. 73+00 (TEST AREA)

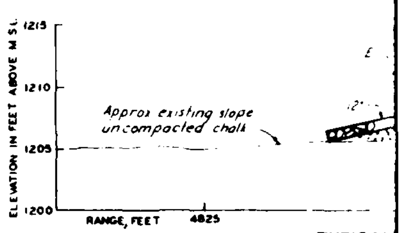


TYPICAL SECTION - DUMPED STONE PROTECTION

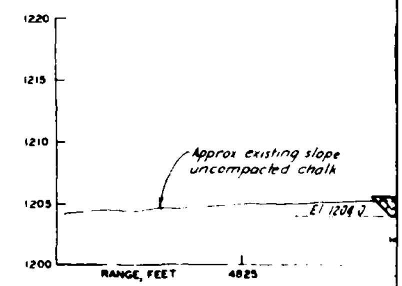
ALTERNATE NO. 2: STA. 36+00 THRU STA. 48+00 AND STA. 73+00 THRU STA. [illegible]
STA. 56+00 THRU STA. 63+00 (TEST AREA-BOULDERS)
STA. 63+00 THRU STA. 68+00 (TEST AREA-QUARRY)



TYPICAL



TYPICAL

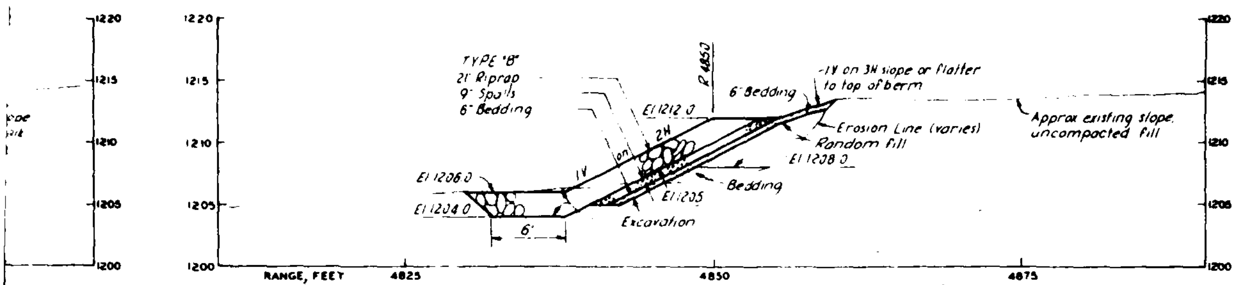


TYPICAL FILL

ALTERNATE NO. 1 STA 36
STA 68

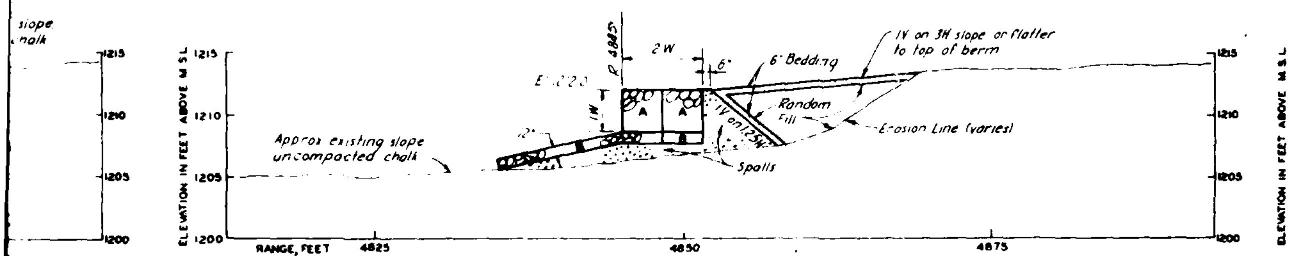
GABION SIZES			
TYPE	LENGTH	WIDTH, W	
A	[illegible]	3' Min. 3'-3" Max.	
B	[illegible]	3' Min. 3'-3" Max.	

EMBANKMENT



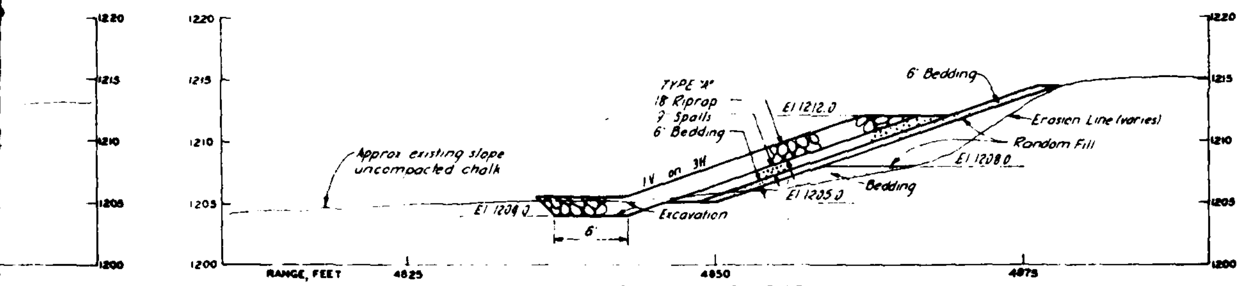
TYPICAL FILL SECTION - 1V ON 2H (TEST AREA)

STA 48+00 THRU STA 53+00



TYPICAL SECTION -GABION PROTECTION (TEST AREA)

BLOCK AND APRON TYPE - STA. 55+50 THRU 58+00



TYPICAL FILL SECTION - 1V ON 3H RIPRAP PROTECTION

ALTERNATE NO. 1 STA 38+00 THRU STA 48+00 & STA. 73+00 THRU STA. 89+00
STA 68+00 THRU STA 73+00 (TEST AREA)

GABION SIZES			
TYPE	LENGTH	WIDTH, W	DEPTH
A	[illegible]	3' Min. 3'-3" Max.	[illegible]
B	[illegible]	3' Min. 3'-3" Max.	1'-0"

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

SCALE 1 INCH = 5 FEET

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

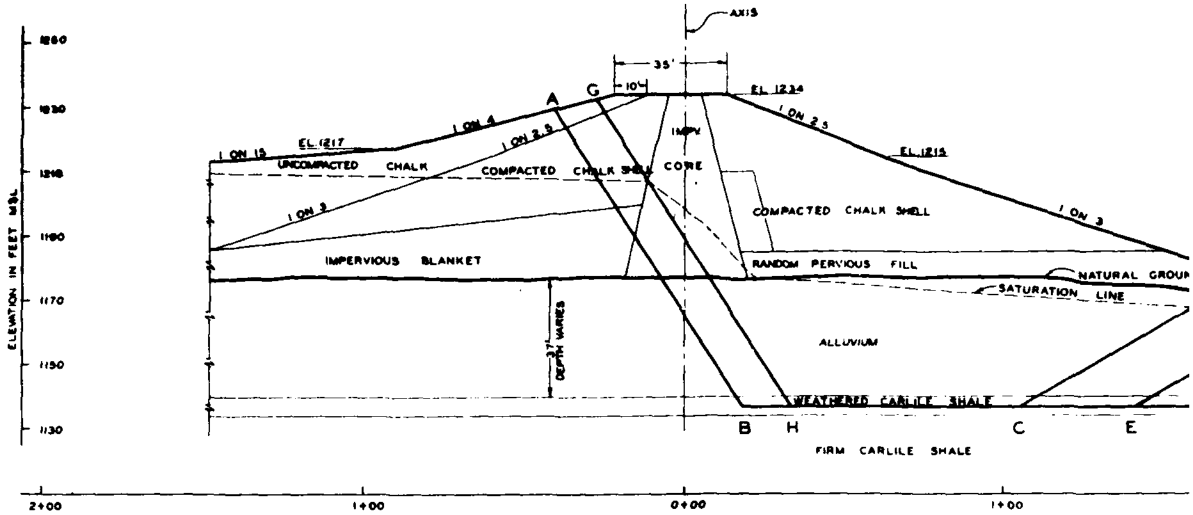
MISSOURI RIVER
GAVINS POINT DAM-LEWIS AND CLARK LAKE
UPSTREAM BERM SLOPE PROTECTION
TYPICAL SECTIONS

DEC 1972

MGI74-40E4

THIS PLAN ACCOMPANIED CONTRACT NO.
MODIFICATION NO.

2



EMBANKMENT SECTION ADJACENT TO SPILLW

SHOWING SLIDE PLANES ANALYZED

SCALE IN FEET

1.

DESIGN ASSUMPTIONS				
MATERIAL	DENSITY T/cu.ft		SHEARING STRENGTH	
	NOT SAT.	SAT.	TAN. φ	C-T/SQ.FT.
DUMPED CHALK	.05	.055	.50	0
COMPACTED CHALK	.055	.061	.50	0
IMPERVIOUS	.062	.064	.35	.35
ALLUVIUM	.062	.064	.60	0
WEA. CARLILE SHALE	.059	.065	.30	.20

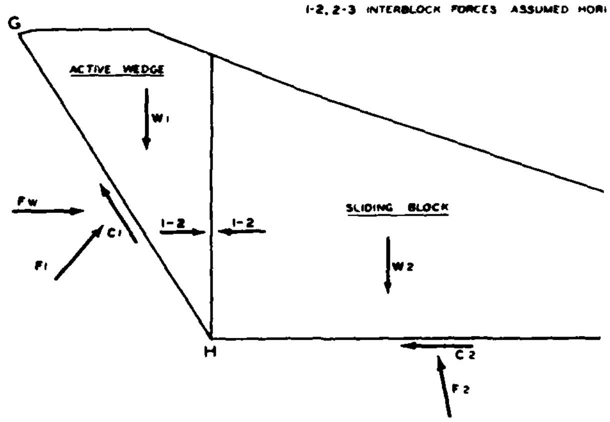
2. THIS ANALYSIS IS FOR A CONDITION AFTER THE COMPLETION OF THE DAM, WITH STEADY SEEPAGE FLOW ESTABLISHED FROM A MAXIMUM NORMAL OPERATING POOL ELEVATION OF 1210.

3. THE EVALUATION OF PORE WATER PRESSURE IN THE EMBANKMENT AND FOUNDATION IS BASED ON THE ASSUMPTION OF HYDROSTATIC PRESSURES BELOW THE ESTIMATED SATURATION LINE.

4. THE ANGLES OF THE SLIDE PLANES FOR THE ACTIVE AND PASSIVE WEDGES WERE BASED ON RANKINES' THEORY, USING WEIGHTED TANGENT φ VALUES.

SYMBOLS.

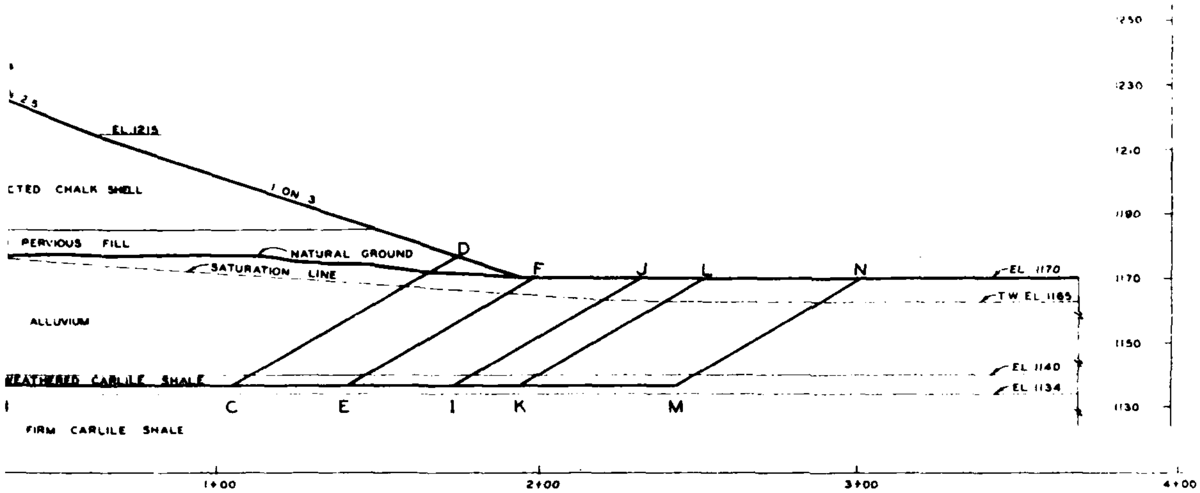
W	EFFECTIVE WEIGHT OF BLOCK CO
C	COHESIVE RESISTANCE ALONG SLID
F	RESULTANT OF FRICTIONAL RESIST TO SLIDE PLANE
Fw	NET HORIZONTAL THRUST FROM
1-2, 2-3	INTERBLOCK FORCES ASSUMED HOR



FORCE DIAGRAM

SLIDE PLANE GHKL

NO SCALE



NT SECTION ADJACENT TO SPILLWAY

/ING SLIDE PLANES ANALYZED

SCALE IN FEET

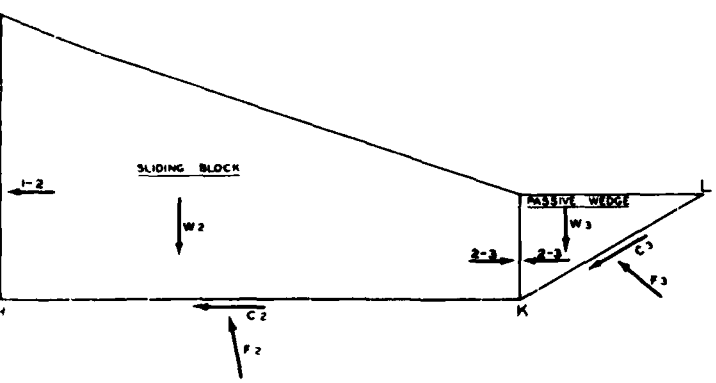
SUMMARY OF RESULTS	
SLIDE PLANE	FACTOR OF SAFETY
ABCD	1.90
ABEF	1.64
ABKL	1.80
GHEF	1.78
GHIJ	1.80
GHKL	1.59
GHMN	1.70

NOTE:

ANALYSIS OF SLIDE PLANES AT HIGHER LEVELS INDICATE THAT THE CRITICAL SLIDE PLANE FOR THIS SECTION WILL PASS THROUGH THE LAYER OF WEATHERED CARLILE SHALE BEDROCK. THE MODIFIED WEDGE METHOD OF ANALYSIS IS USED.

SYMBOLS.

- W — EFFECTIVE WEIGHT OF BLOCK CONSIDERING BOUYANCY
- C — COHESIVE RESISTANCE ALONG SLIDING SURFACE.
- F — RESULTANT OF FRICTIONAL RESISTANCE AND FORCE NORMAL TO SLIDE PLANE
- Fw — NET HORIZONTAL THRUST FROM PORE WATER PRESSURE.
- 1-2, 2-3 — INTERBLOCK FORCES ASSUMED HORIZONTALLY.

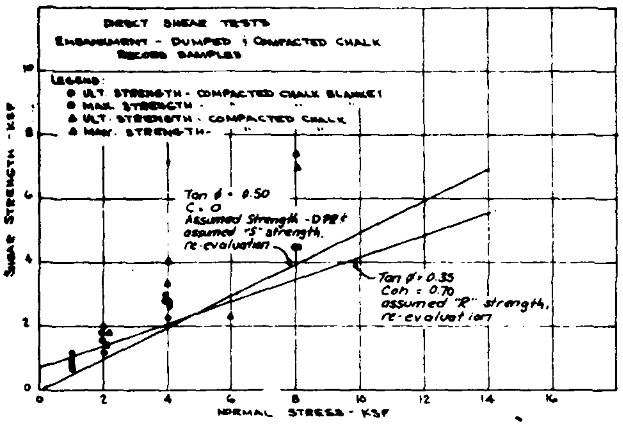


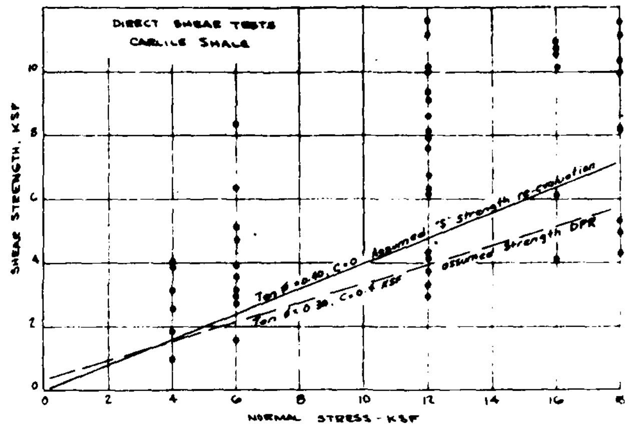
FORCE DIAGRAM

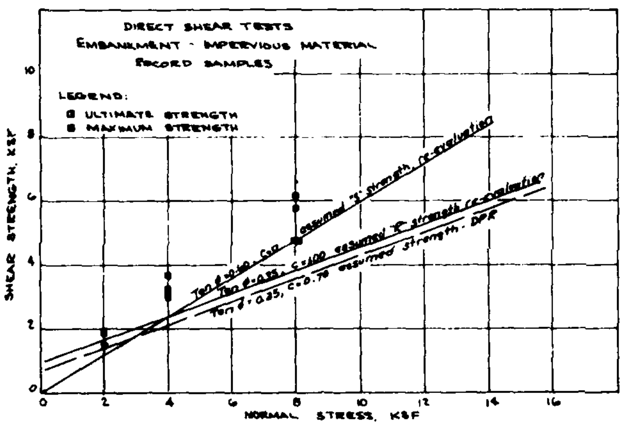
SLIDE PLANE GHKL

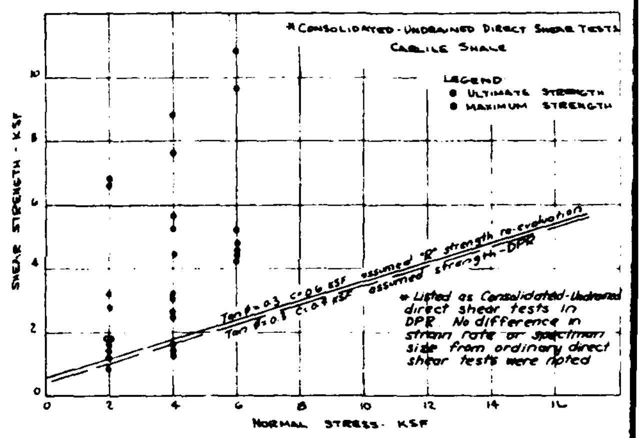
NO SCALE

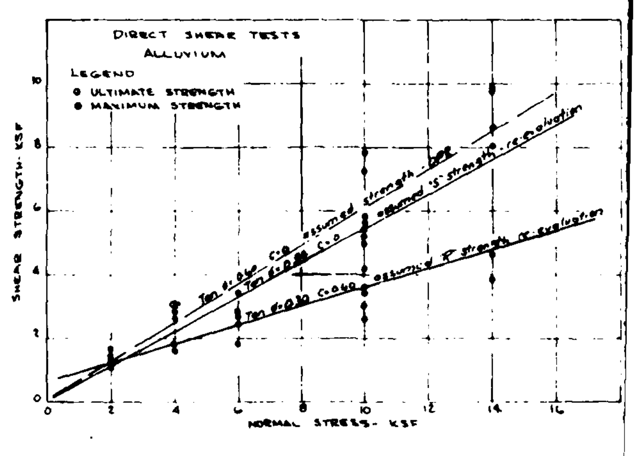
MISSOURI RIVER
GAVINS POINT RESERVOIR
EMBANKMENT STABILITY ANALYSIS
OFFICE OF THE DISTRICT ENGINEER
OMAHA, NEBRASKA FEBRUARY 1952









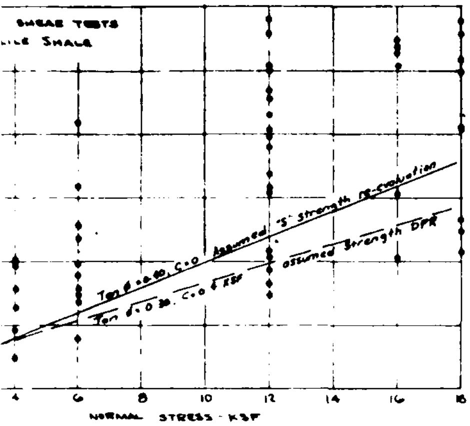


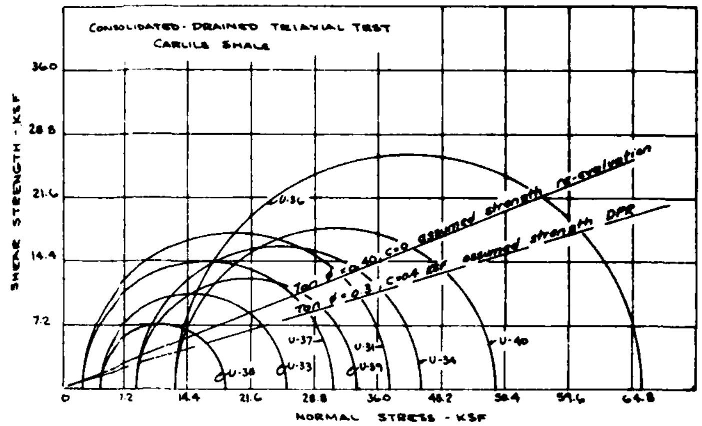
EMBANKMENT MATERIALS							
MATERIAL	MECHANICAL ANALYSIS			ATTERBERG LIMITS		DIRECT SHEAR *	
	GRAVEL	SAND	FINES	LL	PL	C,TSF	TAN ϕ
COMPACTED CHALK	0	13	87	54	32	0.10	0.65
	0	13	87	54	33	0	0.55
	0	12	88	43	26	0.10	0.60
	0	8	92	48	20	0.20	0.47
IMPERVIOUS	0	12	88	36	17	0.60	0.47
	0	39	61	31	18	0.10	0.70

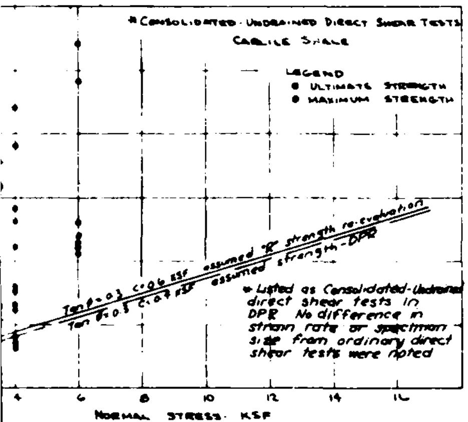
* ULTIMATE STRENGTH

All data shown above are the results of shear tests made on undisturbed samples obtained after construction.

EN



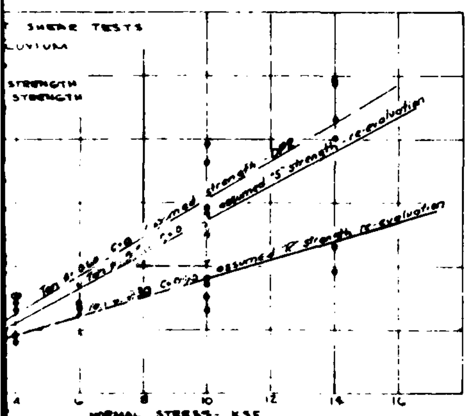




FOUNDATION CARLILE SHALE						
CONSOLIDATED - DRAINED TRIAXIAL TEST *						
SAMPLE NO.	DEPTH FT.	DRY DENSITY lb/ft³	ATTERBERG LIMITS		SHEAR STRENGTH	
			LL	PL	[illegible]	[illegible]
U-31	54.2 - 55.9	115.5	54	21	2.16	37.7
U-33	56.7 - 57.8	110.9	60	25	4.32	25.6
U-34	57.8 - 59.8	113.3	65	21	8.64	41.2
U-36	61.8 - 62.7	112.2	59	20	13.0	65.9
U-37	62.7 - 64.7	122.5	54	17	2.16	30.5
U-38	64.7 - 65.5	120.3	51	20	4.32	18.4
U-39	65.5 - 66.6	121.5	55	21	8.64	33.5
U-40	66.6 - 68.6	119.1	55	23	13.0	49.2

* Samples taken from hole 214

Notes:
1. *Maximum strengths are peak values obtained from stress-strain curves.*
2. *Ultimate strength values are based on 0.3 inch strain from the stress-strain curves.*



THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

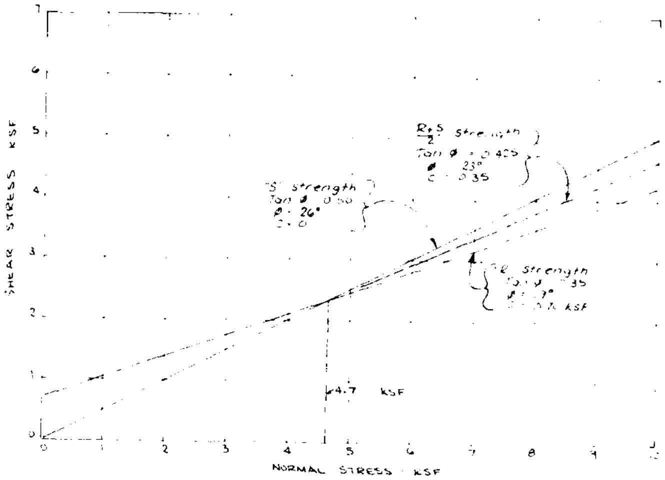
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT DAM
STABILITY RE-EVALUATION
EMBANKMENT AND FOUNDATION
MATERIAL PROPERTIES FOR
DPR AND RE-EVALUATION

SAFETY PAYS

THIS PLAN ACCOMPANIED CONTRACT NO.
MODIFICATION NO.



EMBANKMENT DUMPED AND COMPACTED CHALK

FIGURE 1

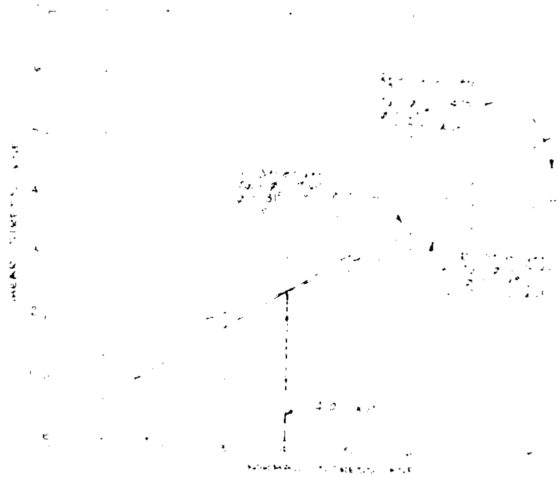
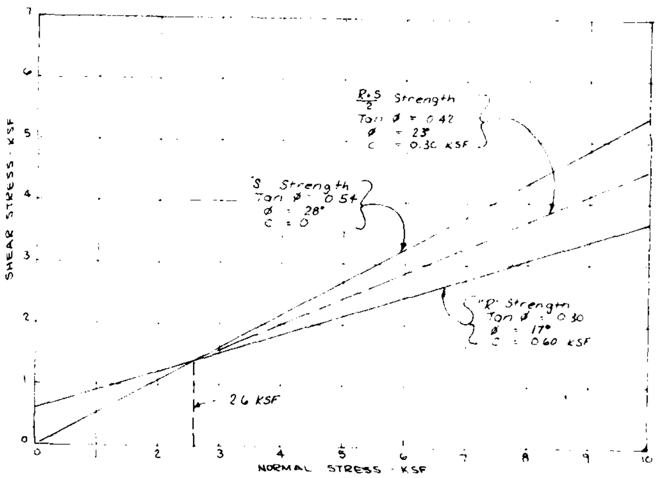


FIGURE 2

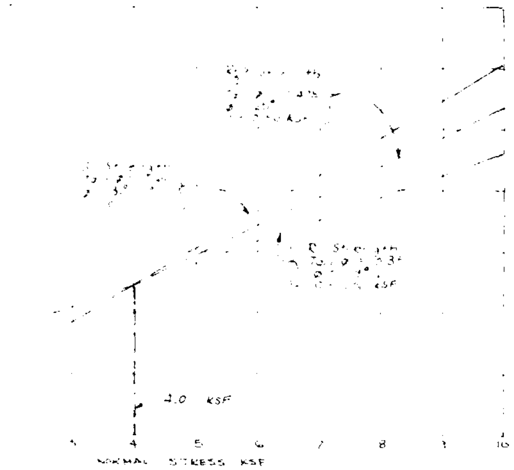


FOUNDATION - ALLUVIUM ZONE

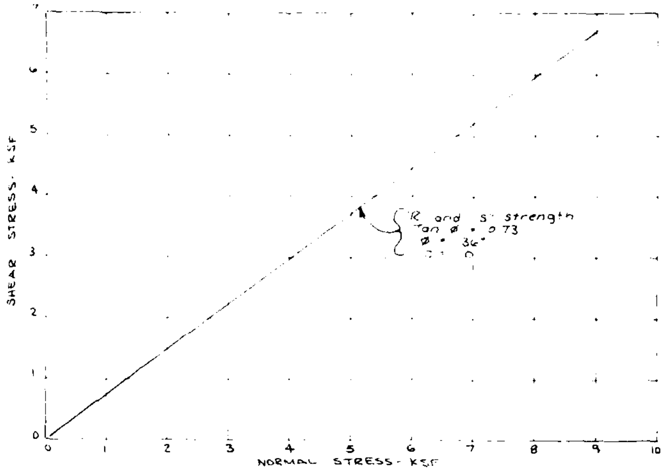
FIGURE 4

FOUNDATION WEATHERED ... SHALE ZONE

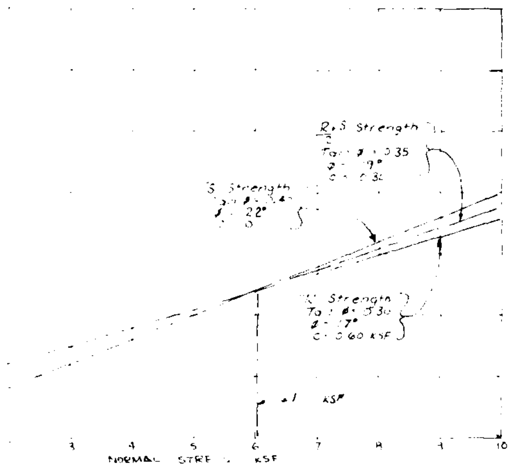
FIGURE 5



EMBANKMENT IMPERVIOUS ZONE
FIGURE 2



EMBANKMENT RANDOM PERVIOUS ZONE
FIGURE 3



FOUNDATION WEATHERED CARLILE SHALE ZONE
FIGURE 5

NOTES:
1. "S" and $\frac{R+S}{2}$ strengths used for Partial Pool and Steady Seepage case
2. "R" and "S" strengths used for Sudden Drawdown case

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT DAM
STABILITY RE-EVALUATION
ADOPTED STRENGTHS
FOR RE-EVALUATION

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

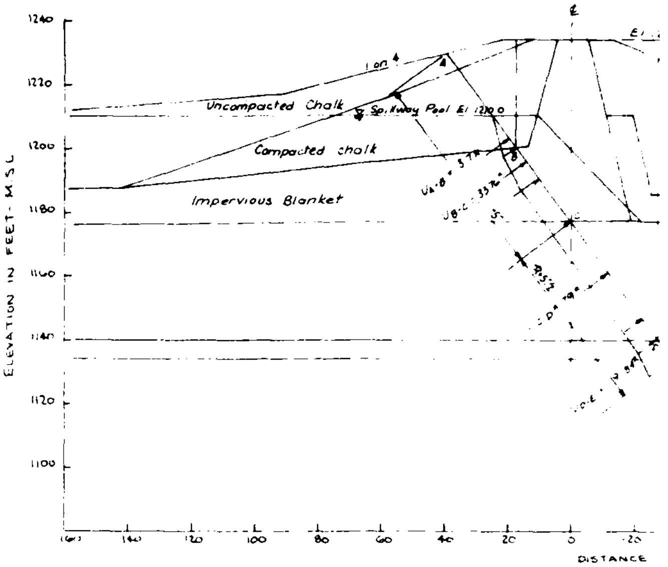
SCALE: AS SHOWN

ADOPTED DESIGN DATA

MATERIAL	UNIT WT (KCF) γ_M	UNIT WT (KCF) γ_S	TAN ϕ R	TAN ϕ S	TAN ϕ $\frac{R+S}{2}$	COHESION (KSF) R	COHESION (KSF) S	COHESION (KSF) $\frac{R+S}{2}$
DUMPED CHALK	0.100	0.110	0.35	0.50	0.425	0.70	0	0.35
COMPACTED CHALK	0.110	0.122	0.35	0.50	0.425	0.70	0	0.35
IMPERVIOUS	0.124	0.128	0.35	0.60	0.475	1.00	0	0.50
ALLUVIUM	0.124	0.128	0.30	0.54	0.42	0.60	0	0.30
WEATHERED CARLILE SHALE	0.118	0.130	0.30	0.40	0.35	0.60	0	0.30
PERVIOUS	0.120	0.130	0.73	0.73	0.73	0	0	0

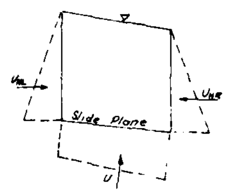
LEGEND
W = Weight
W_h = Earthquake Force
C = Coh. (length)/ trial F.S.
E_A = Resultant force of active wedge
E_{NB} = Resultant force of neutral block
E_P = Resultant force of passive wedge
$\Sigma E = \Sigma E_A + E_{NB} + E_P$
ϕ = Angle of internal friction
ϕ_D = Developed angle of internal friction

U = Uplift



STEADY

Water Forces based on the Following Diagram



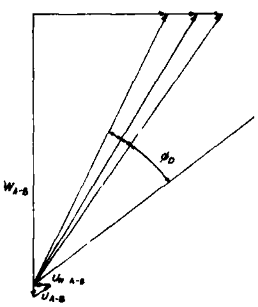
SUBWEDGE VERTICAL SPLICE BOUNDARY

$Wt = (92)(0.100) + (281)(0.110) + (35)(0.122) = 44.4^K \downarrow$
$W_h = 44.4^K \times 0.10 = 4.4^K \rightarrow$
$U_{A-B} = (0.624)(12/2) = 3.7^K$
$U_{HL} - U_{HR} = 3.12^K \leftarrow$

$Wt = (6)(0.100) + (294)(0.110) + (52.5)(0.122) + (240)(0.124) + (260)(0.128) = 102.4^K \downarrow$
$W_h = 102.4^K \times 0.10 = 10.24^K \rightarrow$
$U_{B-C} = \left(\frac{0.624 + 1.25}{2}\right) 12 + \left(\frac{1.25 + 1.40}{2}\right) 17.0 = 33.76^K$
$U_{HL} - U_{HR} = 3.12^K - 15.8^K = 12.68^K \leftarrow$

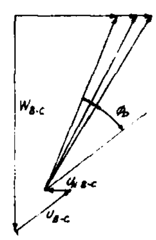
$Wt = (420)(0.124) + \ldots$
$+ (283)(0.120) + \ldots$
$W_h = 244.8^K \times 0.10 \ldots$
$U_{C-D} = \left(\frac{1.40 + 1.90}{2}\right) \ldots$
$U_{HL} - U_{HR} = 15.8^K - 42 \ldots$

TRIAL F.S	TAN ϕ_D	ϕ_D
1.00	0.50	26.6°
1.25	0.40	21.8°
1.50	0.333	18.4°



ACTIVE WEDGE A-B (S)
SCALE 1 INCH = 10 KIPS

TRIAL F.S.	TAN ϕ_D	ϕ_D
1.00	0.60	31.0°
1.25	0.48	25.6°
1.50	0.40	21.8°



ACTIVE WEDGE B-C (S)
SCALE 1 INCH = 30 KIPS

TRIAL F.S	TAN ϕ_D	ϕ_D	C_D
1.0	0.42	22.8°	13.8
1.25	0.336	18.6°	11.0
1.50	0.28	15.6°	9.2

ACTIVE WEDGE
SCALE 1 INCH

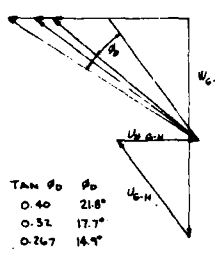
$Wt = (532)(0.110) + (900)(0.120) + (2605)(0.128) + (477)(0.130) = 562^K \downarrow$
$Wh = 562^K \times 0.10 = 56.2^K \rightarrow$
$U_{F-G} = \left(\frac{2.50 + 2.31}{2}\right) 79.5 = 191.2^K \uparrow$

Horizontal Water Force
$U_{HL} - U_{HR} = 49.92^K - 42.71^K = 7.21^K \rightarrow$

TRIAL F.S	C_D	TAN ϕ_D	$E_{F-G} = C_D + (W - U)\,\text{TAN}\,\phi_D - U_H$
1.0	0	0.40	$= 0 + (562 - 191.2)\,0.40 - 7.21 = 141.1^K$
1.25	0	0.32	$= 0 + (562 - 191.2)\,0.32 - 7.21 = 111.45^K$
1.50	0	0.267	$= 0 + (562 - 191.2)\,0.267 - 7.21 = 91.8^K$

CENTRAL BLOCK F-G (S)

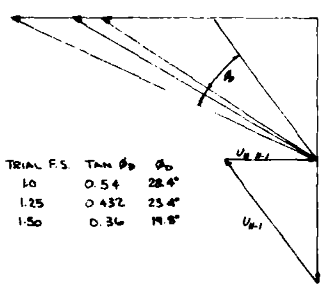
$Wt = (37.5)(0.110) + (217)(0.128) + (22.5)(0.130) = 34.83^K \downarrow$
$W_h = 34.83^K \times 0.10 = 3.48^K \rightarrow$
$U_{G-H} = \left(\frac{2.31 + 1.93}{2}\right) 9 = 19.1^K$
Horizontal Water Force = $12.73^K \rightarrow$



TRIAL F.S	TAN ϕ_D	ϕ_D
1.0	0.40	21.8°
1.25	0.32	17.7°
1.50	0.267	14.9°

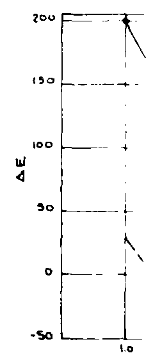
PASSIVE WEDGE G-H (S)
SCALE: 1 INCH = 10 KIPS

$Wt = (24)(0.110) + (635.5)(0.128) = 84^K \downarrow$
$W_h = 84^K \times 0.10 = 8.4^K \rightarrow$
$U_{H-I} = (1.93)(51/2) = 49.2^K$
Horizontal Water Force = $30^K \rightarrow$



TRIAL F.S.	TAN ϕ_D	ϕ_D
1.0	0.54	28.4°
1.25	0.432	23.4°
1.50	0.36	19.8°

PASSIVE WEDGE H-I (S)
SCALE 1 INCH = 20 KIPS



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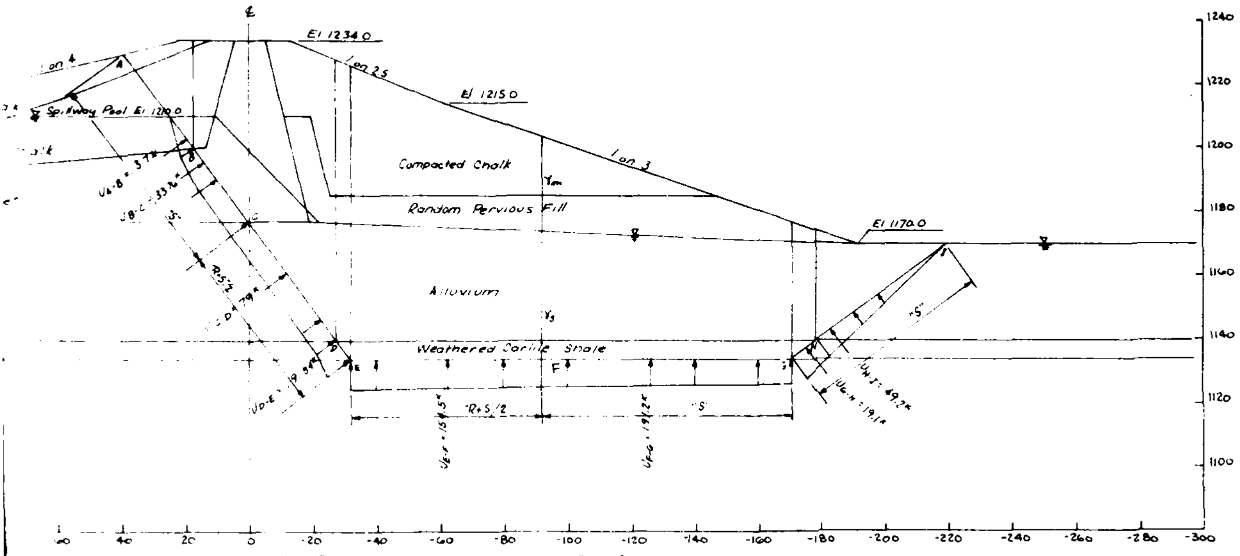


STEADY SEEPAGE - STATION 30+00

SCALE 1 INCH = 20 FEET

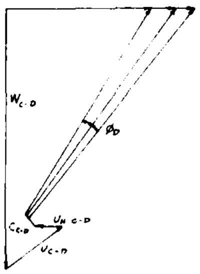
$W_t = (420)(0.124) + (255)(0.128) + (540)(0.110)$
$\quad + (283)(0.120) + (12)(0.130) + (509)(0.128) = 244.8^K \downarrow$

$W_h = 244.8^K \times 0.10 = 24.48^K \rightarrow$

$U_{C-D} = \left(\frac{1.40 + 1.90}{2}\right) 37 + \left(\frac{1.90 + 2.31}{2}\right) 8.5 = 79^K$

$U_{HL} - U_{HR} = 15.8^K - 42.7^K = 26.9^K \leftarrow$

TRIAL F.S.	TAN ϕ_D	ϕ_D	C_D
1.0	0.42	22.8°	13.8
1.25	0.336	18.6°	11.0
1.50	0.28	15.6°	9.2



ACTIVE WEDGE C-D (R+S/2)

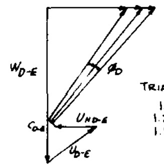
SCALE 1 INCH = 60 KIPS

$W_t = (189)(0.110) + (383)(0.120)$
$\quad + (166.5)(0.128) + (15)(0.130) = 48.64^K \downarrow$

$W_h = 48.64^K \times 0.10 = 4.86^K \rightarrow$

$U_{D-E} = \left(\frac{2.31 + 2.65}{2}\right) 8 = 19.84^K$

$U_{HL} - U_{HR} = 42.7^K - 56.3^K = 13.6^K \leftarrow$



TRIAL F.S.	TAN ϕ_D	ϕ_D	C_D
1.0	0.35	19.3°	2.4
1.25	0.28	15.6°	1.92
1.50	0.233	13.1°	1.60

ACTIVE WEDGE D-E (R+S/2)

SCALE 1 INCH = 20 KIPS

$W_t = (1760)(0.110) + (585)(0.120) +$
$\quad (2145)(0.128) + (360)(0.130) = 585.16^K \downarrow$

$W_h = 585.16^K \times 0.10 = 58.5^K \rightarrow$

$U_{E-F} = \left(\frac{2.65 + 2.50}{2}\right) 60 = 154.5^K \uparrow$

HORIZONTAL WATER FORCE

$U_{HL} - U_{HR} = 56.35^K - 49.92^K = 6.43^K \rightarrow$

TRIAL F.S.	TAN ϕ_D	C_D
1.0	0.35	18
1.25	0.28	14.4
1.50	0.23	12

$E_{E-F} = C_D + (W-U) \tan\phi_D - U_H$

$18 + (585.16 - 154.5)\ 0.35 - 6.43 = 162.3^K$

$14.4 + (585.16 - 154.5)\ 0.28 - 6.43 = 128.5^K$

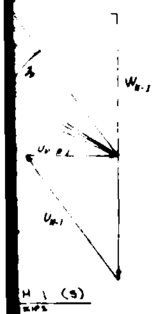
$12 + (585.16 - 154.5)\ 0.233 - 6.43 = 106.0^K$

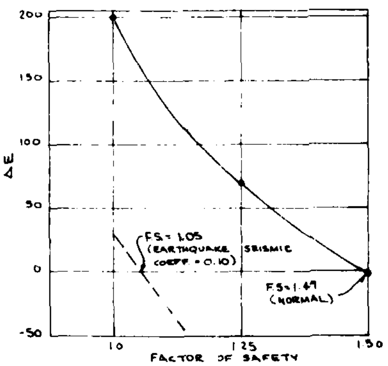
CENTRAL BLOCK E-F (R+S/2)

THIS [redacted]
THREE-EIGHT[redacted]

SUMMARY OF FORCES											
TRIAL FS	A-B	B-C	C-D	D-E	E-F	F-G	G-H	H-I	E_A	E_P	ΔE
1.0	→ 21 (25.4)*	→ 48 (58.24)	→ 135 (159.5)	→ 24 (28.9)	← 162.3 (103.8)	← 141.1 (84.9)	← 29 (25.5)	← 97 (88.6)	→ 228 (272)	← 430 (302.8)	← 202 (30.8)*
1.25	26 (30.4)	56 (68.24)	158 (182.5)	29 (33.9)	128.5 (70)	111.5 (95.3)	24 (20.5)	78 (69.6)	271 (315)	342 (215.4)	← 71 → (99.6)*
1.50	28 (32.4)	63 (73.24)	173 (197.5)	32 (36.9)	106 (47.5)	91.8 (35.6)	22 (18.5)	72 (62.6)	296 (340)	292 (168.2)	4 (174.8)*

* CONSIDERED EARTHQUAKE





FACTOR OF SAFETY DETERMINATION

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT DAM
STABILITY RE-EVALUATION
STEADY SEEPAGE FROM SPILLWAY CREST
CRITICAL SECTION - STA. 30+00
(MANUAL PROCEDURES)

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

TABLE I - SUMMARY OF RELIEF WELL SPACING & DISCHARGE COMPUTATIONS

	Locations			
	Station 27+00 to 41+00	Station 41+00 to 51+00	Station 51+00 to 73+00	Station 73+00 to 91+00
Av. Disch. Elev.	1184	1174	1177	1169
Av. Bedrock Elev.	1110	1050	1045	1110
Av. Bottom Well Elev.	1110	1075	1075	1110
Total Head, H	38.3	48.3	45.3	53.3
Thickness of downstream blanket Z_b	12.0	5.0	5.0	5.0
Allowable Uplift = 0.84 Z_b	10.1	4.2	4.2	4.2
Thickness of Substratum, D	62	119	127	53
Transformed, D^1	-	238	254	-
Depth of Well, Z_w	Full Pene.	94	97	Full Pene.
Transformed, Z_w^1	-	188	194	-
Upstream Head Loss h_1	30.1	44.2	41.5	49.8
Upstream Gradient, S_u	.0310	.0456	.0428	.0514
Downstream Gradient, S_d	.0109	.0102	.0095	.0087
Net Gradient, S	.0201	.0354	.0333	.0427
Mean Potential, P_a	8.11	4.05	3.82	3.61
Midpoint Potential P_m	9.22	4.33	4.21	4.27
Computed Well Spacing	*500+	140	140	140
Est. disch. per well in cfs	0.62	1.18	1.18	0.63

Notes:

1. See Plate 4 for definition of symbols.
2. Upstream Resistance d = 970 ft.
3. Downstream Resistance X_1
   Sta. 26+40 to Sta. 41+00 = 750 ft.
   Sta. 41+00 to Sta. 91+00 = 400 ft.
4. Radius of Relief Well = 0.5 ft.
5. Maximum Pool Elev. = 1222.3 m.s.l.
6. *Spacing arbitrarily reduced to 250 feet.
7. Wells arbitrarily provided at 100-foot spacings within closure area from Sta. 91+00 to Sta. 97+50.

TABLE I - SUMMARY OF RELIEF WELL SPACING & DISCHARGE COMPUTATIONS

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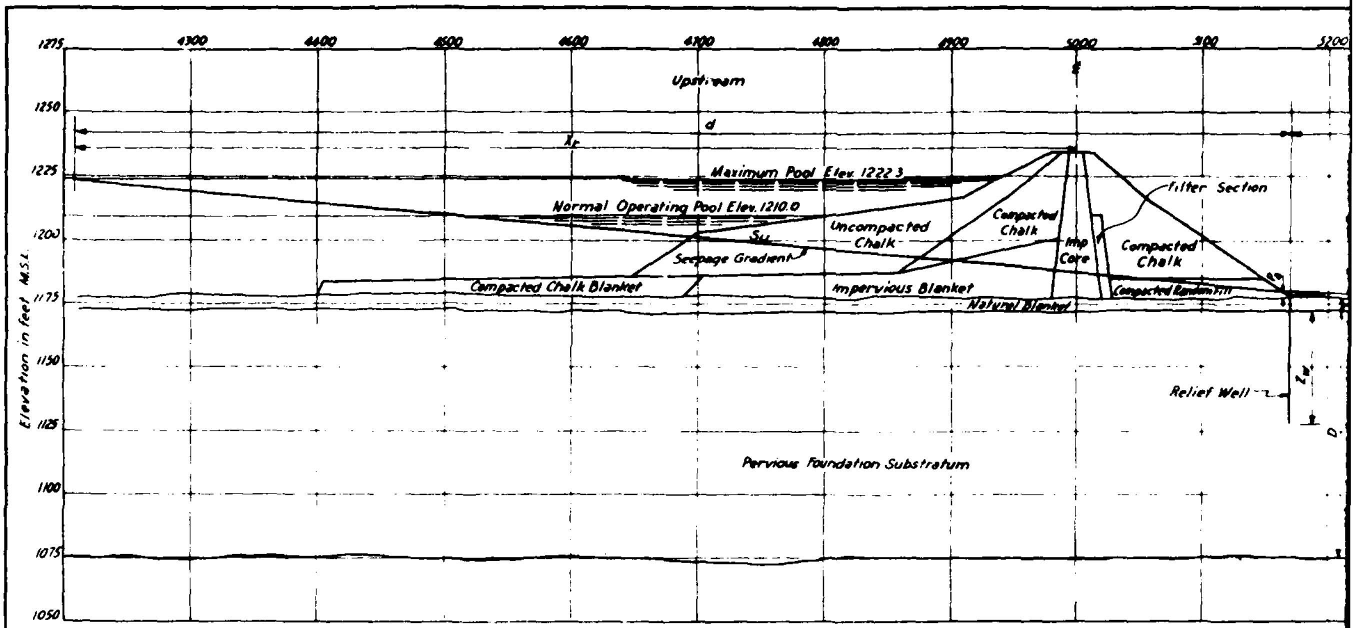
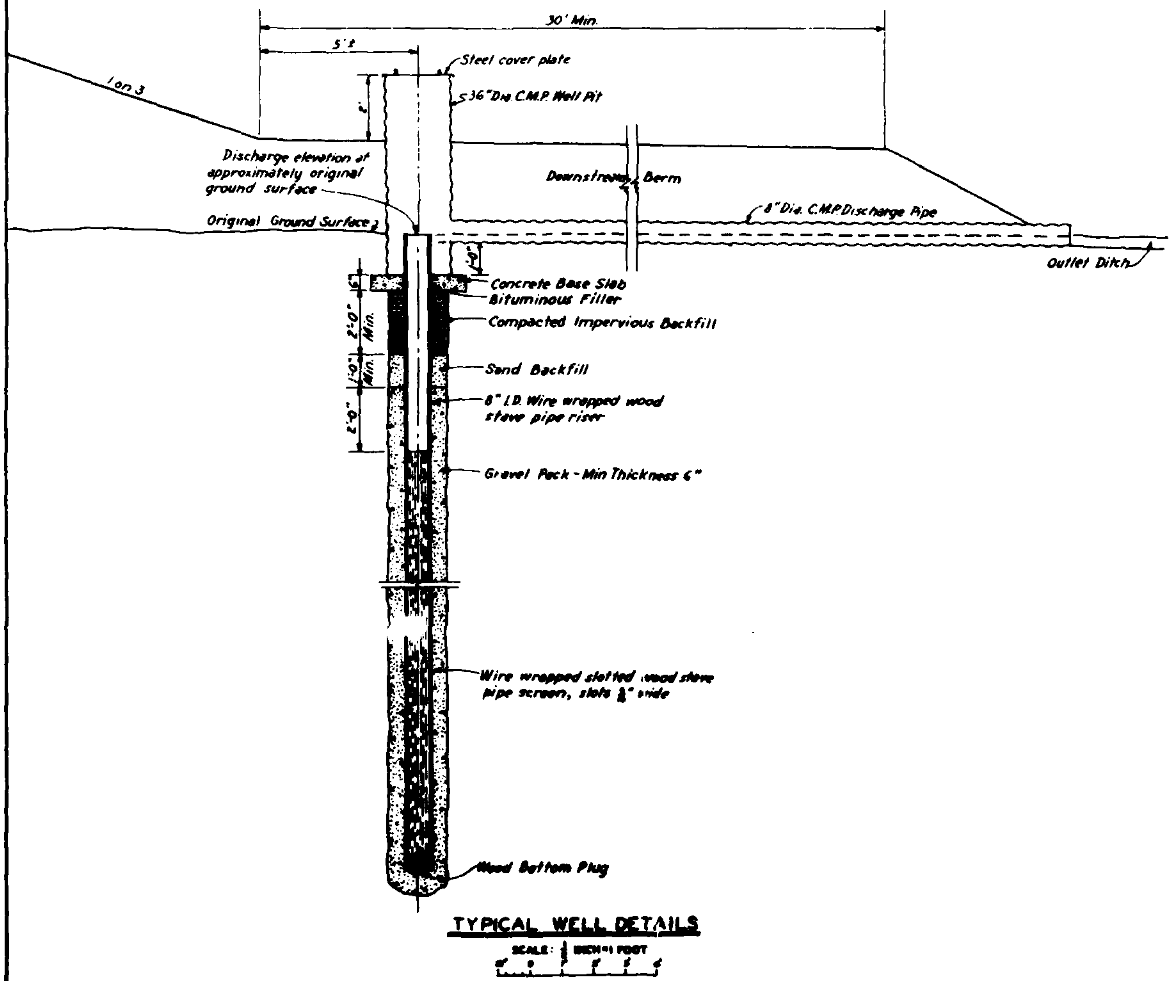
CORPS OF ENGINEERS



TYPICAL SECTION ILLUSTRATING NOMENCLATURE



TYPICAL WELL DETAILS

SCALE: ½ INCH = 1 FOOT

GENERA

1. X_r = 800 ft
2. d = 970 ft.
3. X_d
4. K_f = 0.002 f
5. Weight of so the allowa ground sur
6. See paragr
7. Design max
8. Assume rea
9. Ratio of ho foundation
10. In the case transforme

TYPICAL

DESIGN DATA

Average gr
Average be
Bottom ele
Range of lin
Total upstre
Total head,
Depth of su
Depth of we
Transforme
Transform

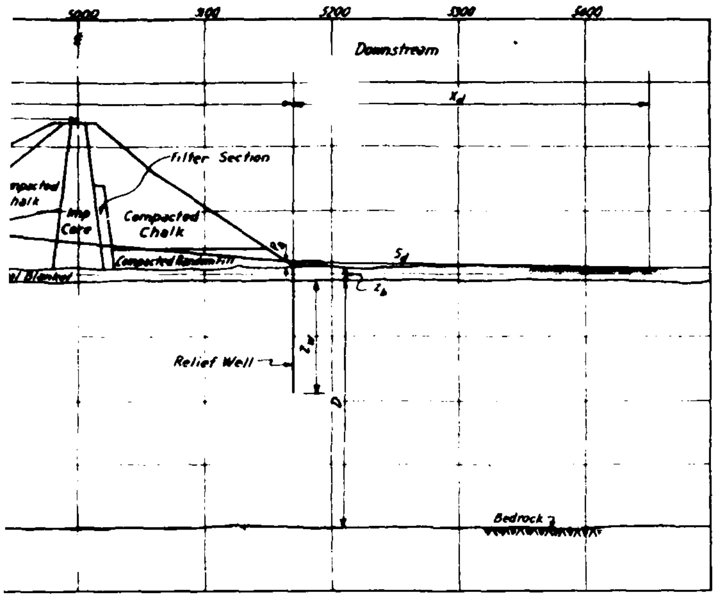
WELL SPACIN

Assume wel trial and er uplift

a (Assumed
a (Assumed
h_1
S_u
S_d
S
P_a/S
P_a
P_m/S
P_m

The comp equal to t well spaci

U. S. ARMY



OMENCLATURE

NOMENCLATURE

Z_b = Thickness of relatively impervious downstream blanket.
D = Thickness of pervious substratum.
Z_w = Depth of relief wells.
H = Total head.
P_a = Mean uplift pressure over plane of wells.
P_m = Surface uplift pressure at midpoint between wells.
h_1 = $H - P_a$ = Mean total headless from source to line of wells.
d = Total effective resistance upstream of line of wells.
X_r = Effective resistance upstream of axis of dam.
X_1 = Effective resistance downstream line of wells.
S_u = h_1/d = Mean gradient upstream of wells.
S_d = P_a/X_1 = Mean gradient downstream of wells.
S = $S_u - S_d$ = Net gradient producing discharge from wells.
a = Well spacing.
K_f = Horizontal permeability of pervious substratum.
K_v = Vertical permeability of pervious substratum.
Q_w = $K_f Z_w a S$ = Discharge from well.
r_w = Radius of well.

WELL SPACING FORMULAE

1. Mean pressure formula:

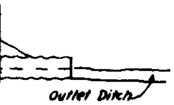
$$\frac{P_a}{S} = \frac{aD}{2\pi Z_w} \log_e \frac{a}{2\pi r_w} + 0.11a\left(\frac{D}{a} - 1\right)\left(\frac{D}{Z_w} - 1\right)$$

2. Mid-point surface pressure formula:

$$\frac{P_m}{S} = \frac{aD}{2\pi Z_w} \log_e \frac{a}{2\pi r_w} + 0.11a$$

GENERAL DESIGN ASSUMPTIONS & DATA

1. X_r = 800 ft (See paragraph 11)
2. d = 970 ft.
3. X_d (See paragraph 10)
4. K_f = 0.002 ft/sec.
5. Weight of saturated blanket material = 115 lb/cu.ft., therefore the allowable uplift measured in feet of water above the ground surface is 0.84 Z_b for a factor of safety of 1.0.
6. See paragraph 8 for assumed thickness of downstream blanket.
7. Design maximum pool water surface elevation = 1222.3
8. Assume radius of well, r_w = 0.5 ft.
9. Ratio of horizontal permeability to vertical permeability of foundation material is in the order of 4 to 1.
10. In the case of partial penetrating wells the depths are transformed in accordance with $\sqrt{K_f/K_v} = \sqrt{4} = 2$



TYPICAL WELL SPACING COMPUTATION

STA. 51+00 TO STA. 73+00
PARTIAL PENETRATION WELLS

DESIGN DATA

Average ground surface elevation - - - - - - - - 1177
Average bedrock elevation - - - - - - - - - - 1045
Bottom elevation of wells - - - - - - - - - - - 1075
Range of line of relief wells - - - - - - - - - - 5170
Total upstream resistance, d - - - - - - - - - - 970
Total head, H - - - - - - - - - - - - - - - - 45.3
Depth of substratum, D - - - - - - - - - - - - 127
Depth of well, Z_w - - - - - - - - - - - - - - 97
Transformed depth of substratum, D' - - - - - - - 254
Transformed depth of well, Z_w' - - - - - - - - 194

WELL SPACING COMPUTATIONS

Assume well spacing "a" and mean pressure "P_a". Compute by trial and error midpoint pressure "P_m" which is = allowable uplift.

	1st Trial	2nd Trial	3rd Trial
P_a (Assumed)	4.0	3.6	3.8
a (Assumed)	120	120	140
h_1	41.3	41.7	41.5
S_u	.0426	.0430	.0428
S_d	.0100	.0090	.0095
S	.0326	.0340	.0333
P_a/S	95.8	95.8	114.7
P_a	3.12	3.26	3.82
P_m/S	104.4	104.4	126.2
P_m	3.41	3.55	4.21

The computed midpoint pressure in the 3rd trial is equal to the allowable uplift pressure, therefore a well spacing of 140 ft. is adequate for this area.

THIS DRAWING HAS BEEN REDUCED TO THREE EIGHTHS THE ORIGINAL SCALE

SAFETY PAYS

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

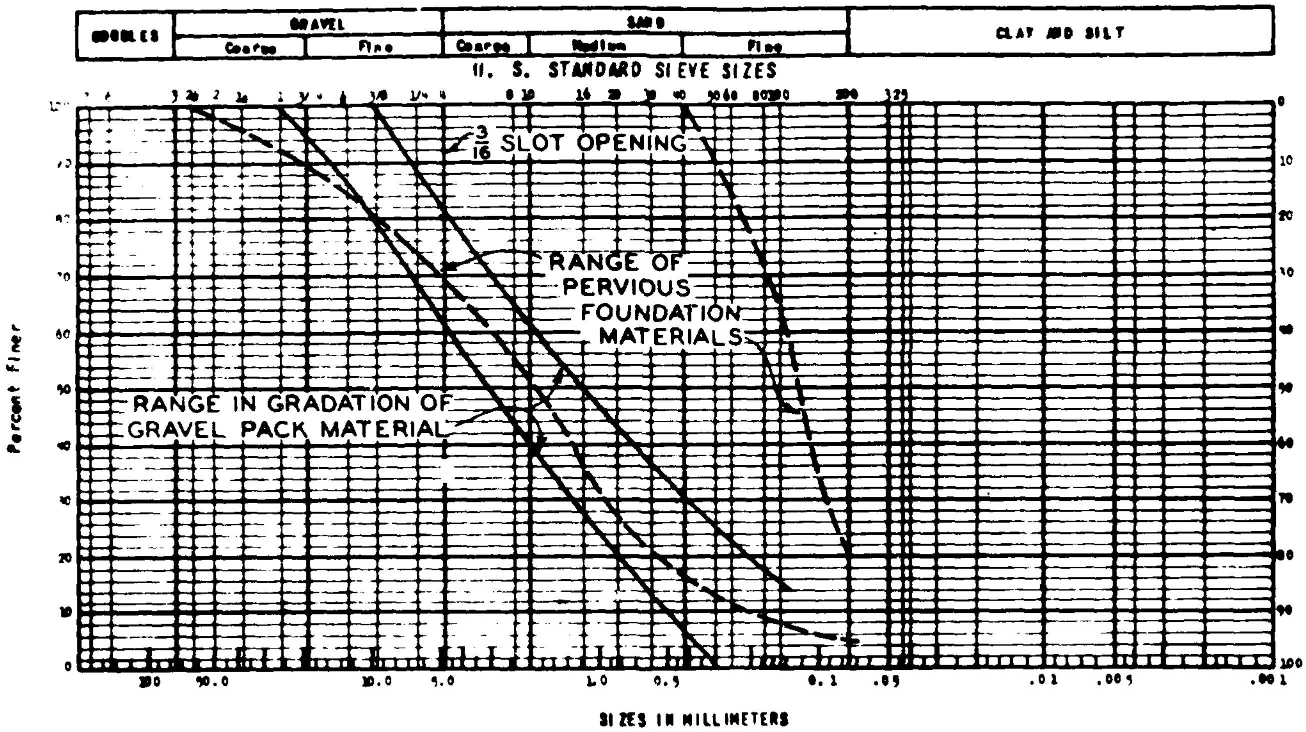
MISSOURI RIVER
GAVINS POINT RESERVOIR
PRESSURE RELIEF WELLS
TYPICAL WELL SPACING
COMPUTATIONS AND DETAILS

DESIGNED BY: C.H.H.
DRAWN BY: C.H.H.
TRACED BY: W.C.H.
CHECKED BY: G.P.A.
DATE: DEC. 1954

Army-CE-Omaha, Nebr.

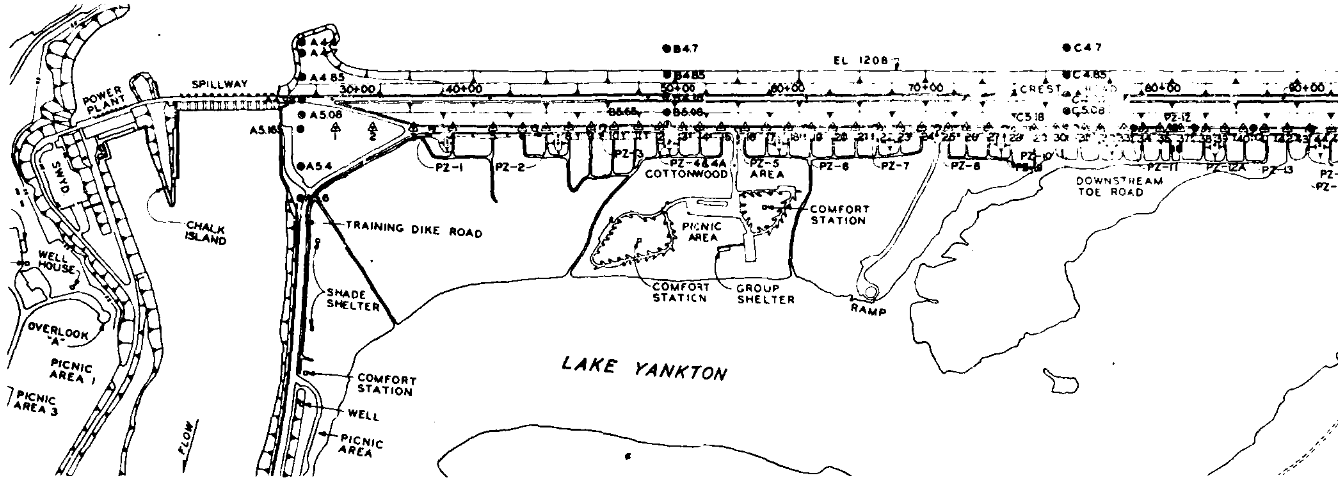
THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-066-eng . MODIFICATION NO.

2

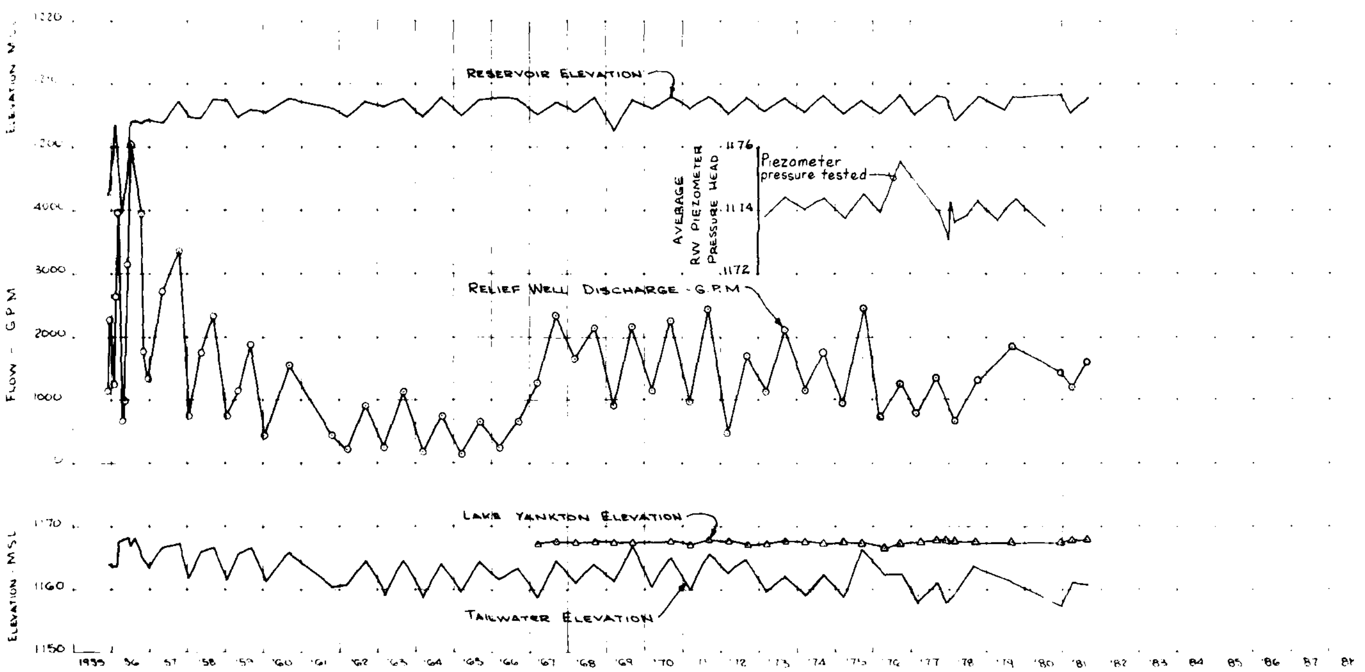


MISSOURI RIVER
GAVINS POINT RESERVOIR
PRESSURE RELIEF WELLS
GRADATION CURVES
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT OMAHA, NEBRASKA
NOV. 1954

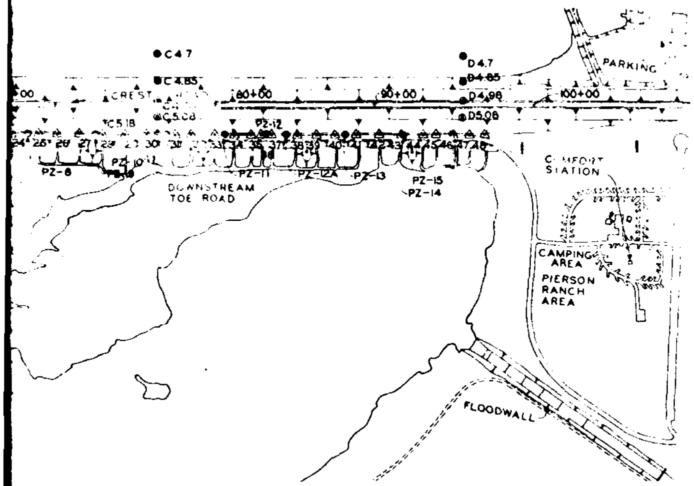
Army-CE-Omaha, Nebr.

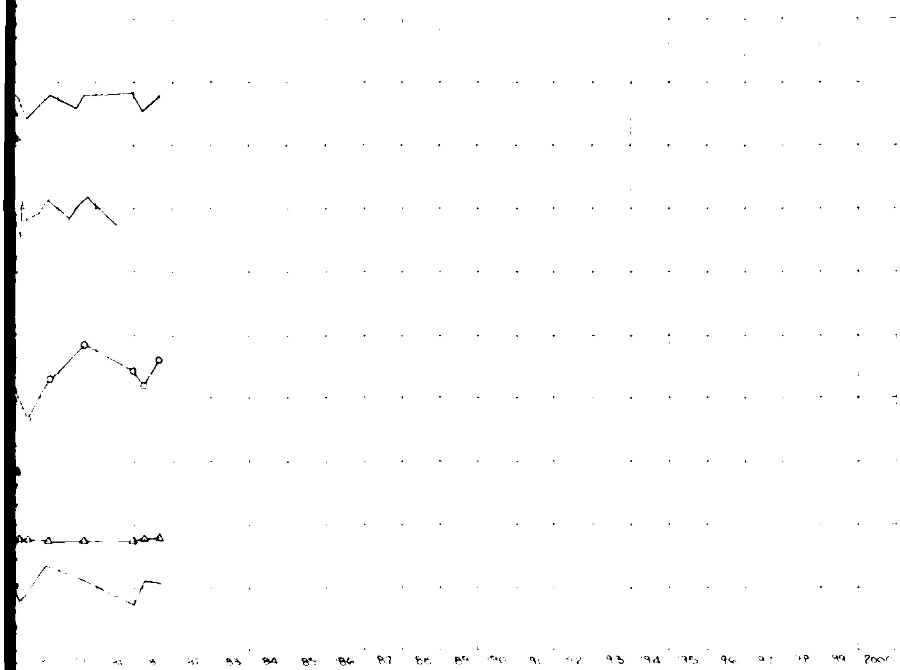


POWER PLANT
SPILLWAY
DAMS
CHALK ISLAND
WELL HOUSE
OVERLOOK "A"
PICNIC AREA 1
PICNIC AREA 3
FLOW
TRAINING DIKE ROAD
SHADE SHELTER
COMFORT STATION
WELL
PICNIC AREA
LAKE YANKTON
EL 1208
CREST
COTTONWOOD
PZ-5 AREA
PICNIC AREA
COMFORT STATION
GROUP SHELTER
RAMP
DOWNSTREAM TOE ROAD



Elevation MSL
Reservoir Elevation
Average RW Piezometer Pressure Head
Piezometer pressure tested
Relief Well Discharge - G.P.M.
Flow - G P M
Lake Yankton Elevation
Tailwater Elevation
Elevation MSL





THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
CHIEF SECTION
RECOMMEND:
CHIEF BRANCH

MISSOURI RIVER
GAVINS POINT DAM
EMBANKMENT
RELIEF WELL OBSERVATIONS
TOTAL FLOW

APPROVED: DATE:
CHIEF ENGINEERING DIVISION
APPROVED:
COL. C. E., DISTRICT ENGINEER
SCALE: AS SHOWN
SPEC. NO.
DRAWING NUMBER
SHEET

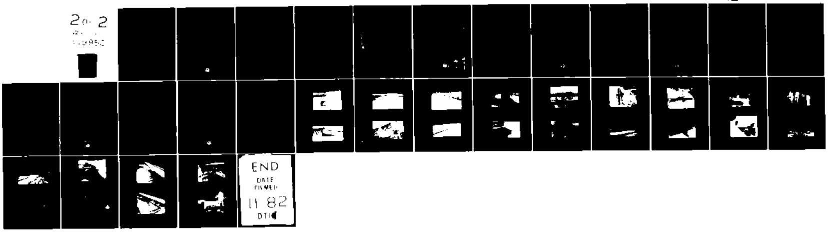
SAFETY PAYS

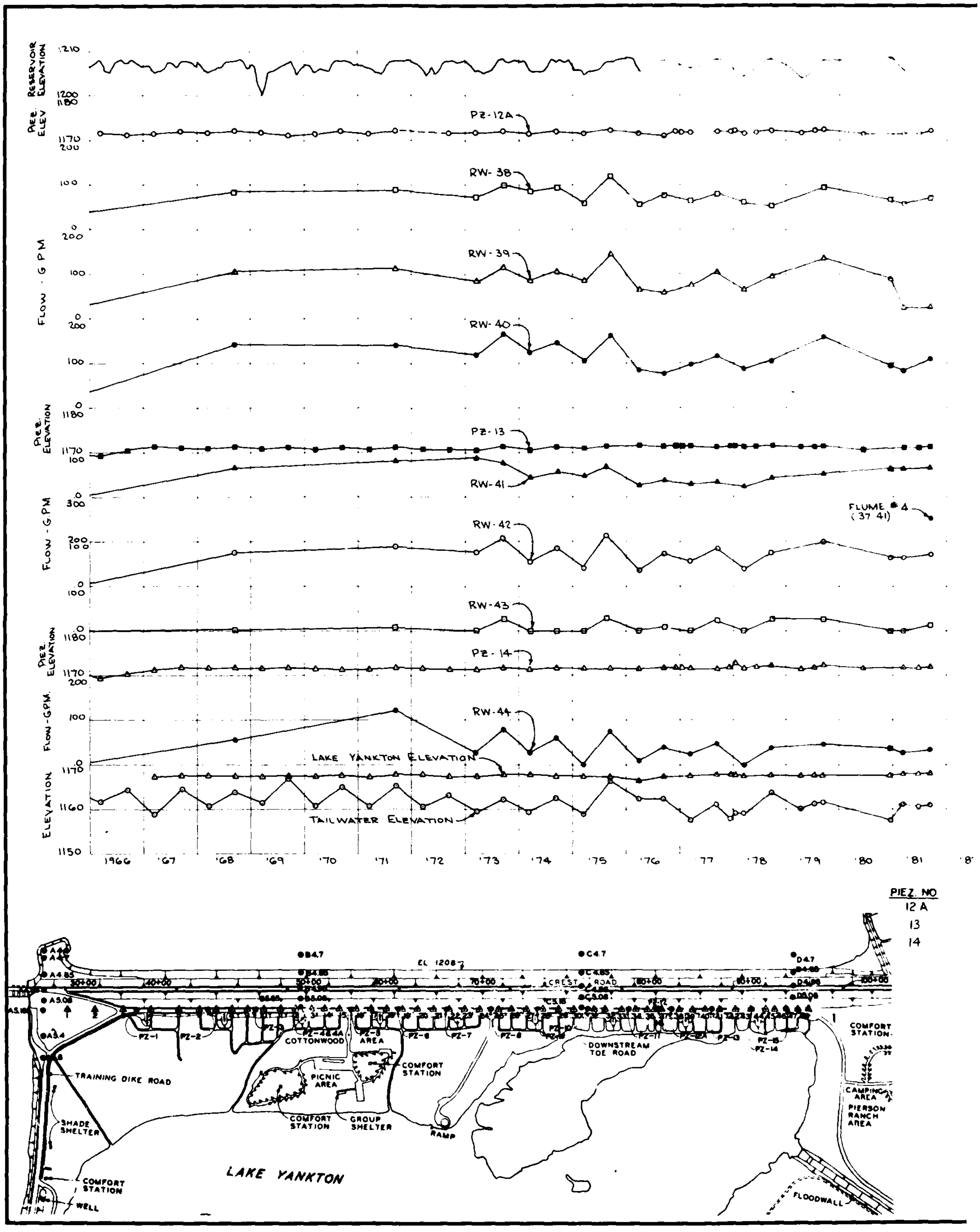
THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

AD-A119 952 ARMY ENGINEER DISTRICT OMAHA NEBR F/G 13/13
MISSOURI RIVER, GAVINS POINT DAM - LEWIS AND CLARK LAKE, NEBRAS--ETC(U)
MAY 82

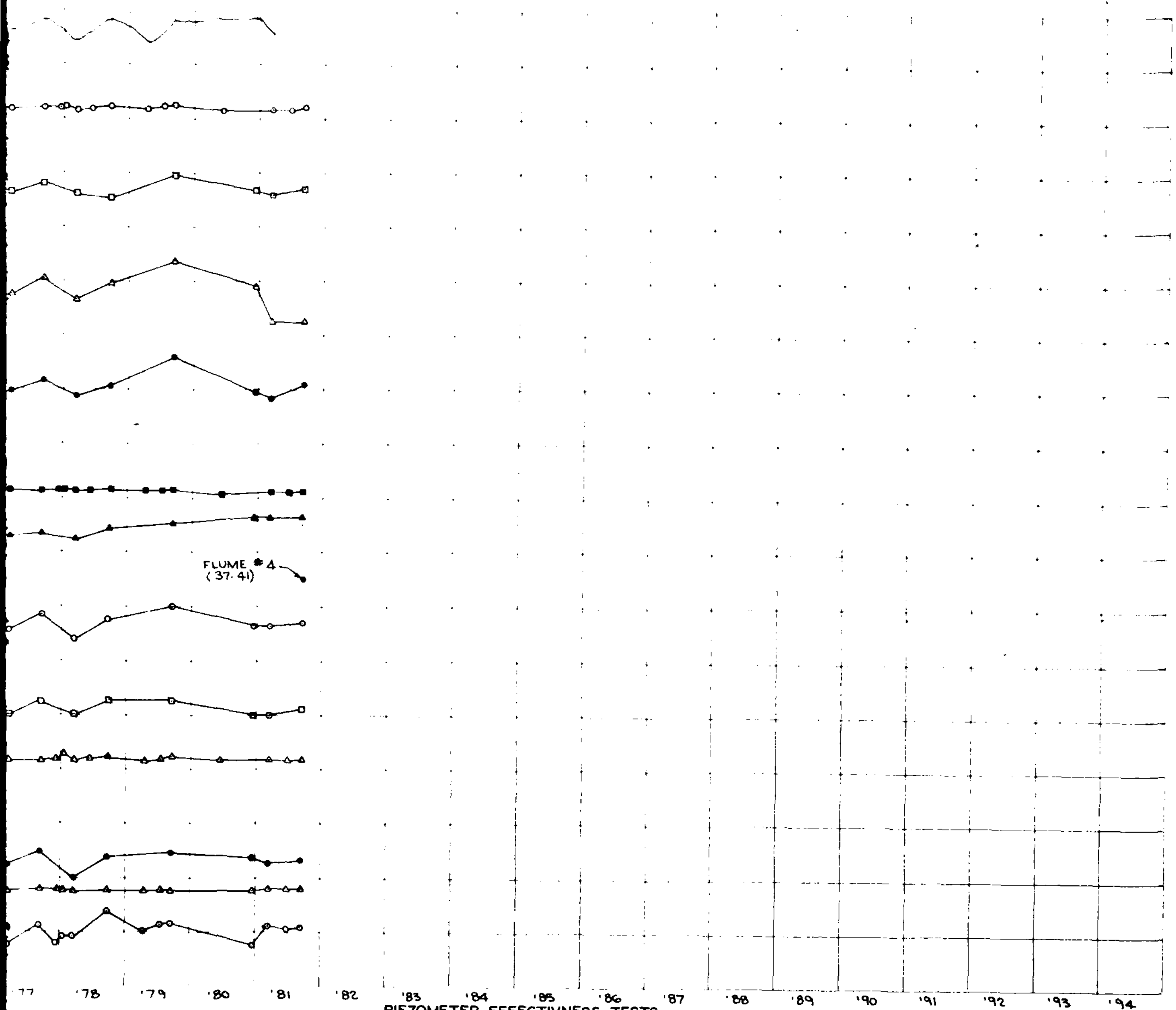
UNCLASSIFIED NL

2 of 2





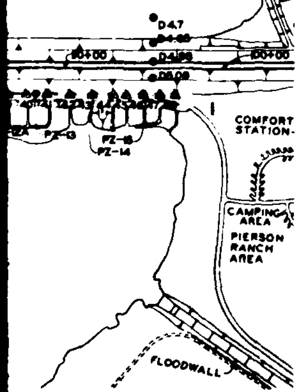
RESERVOIR ELEVATION
1210
1200
PIEZ. ELEV
1180
1170
PZ-12A
FLOW - G P M
200
100
0
RW-38
RW-39
RW-40
PIEZ. ELEVATION
PZ-13
RW-41
300
FLUME # 4 (37 41)
RW-42
RW-43
PZ-14
FLOW-GPM.
RW-44
LAKE YANKTON ELEVATION
TAILWATER ELEVATION
ELEVATION
1160
1150
1966
'67
'68
'69
'70
'71
'72
'73
'74
'75
'76
'77
'78
'79
'80
'81
PIEZ. NO
12 A
13
14
EL 1208
CREST ROAD
DOWNSTREAM TOE ROAD
COTTONWOOD
AREA
COMFORT STATION
PICNIC AREA
GROUP SHELTER
RAMP
TRAINING DIKE ROAD
SHADE SHELTER
LAKE YANKTON
WELL
CAMPING AREA
PIERSON RANCH AREA
FLOODWALL



PIEZOMETER EFFECTIVNESS TESTS

PIEZ. NO.	DATE	RESULTS	ACTION
12 A	Aug. 10-20, 1976	Near full recovery ∠ 10 days	Rehab. Nov. '76
13	"	OK	OK
14	"	Near full recovery ∠ 10 days	Rehab. Nov. '76

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.



SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

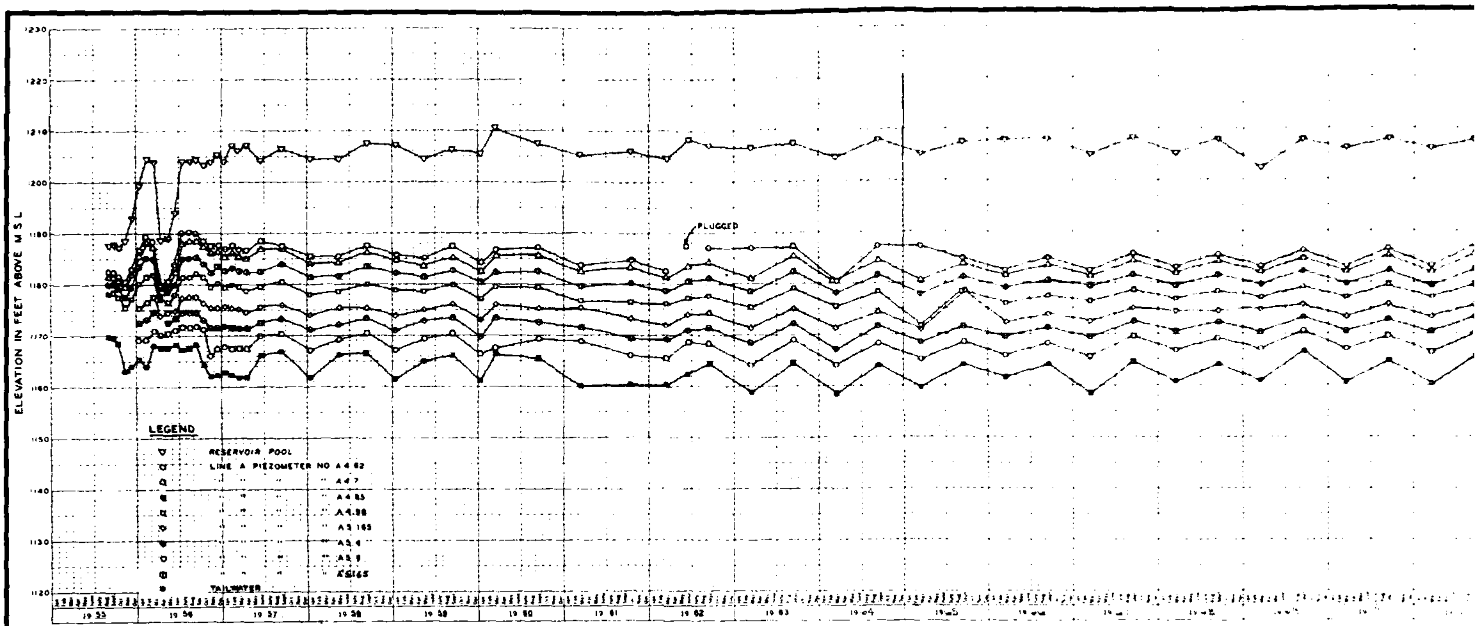
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

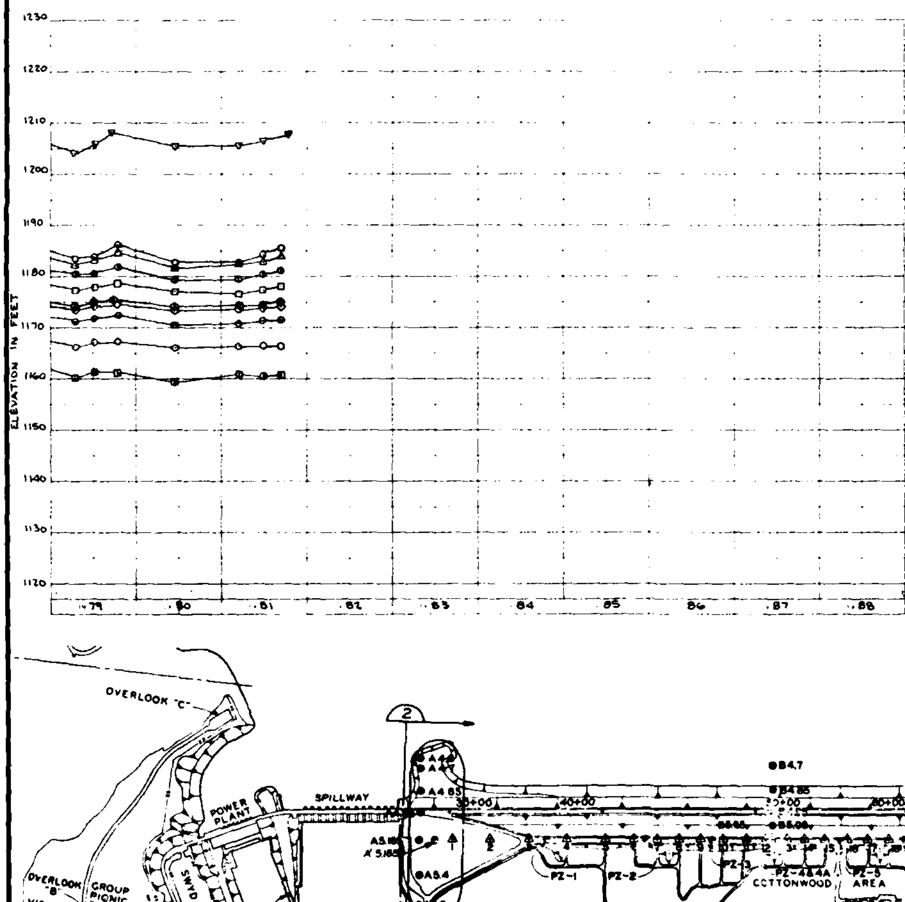
MISSOURI RIVER
GAVINS POINT DAM
EMBANKMENT
RELIEF WELL AND RELIEF WELL PIEZ OBSERVATIONS

CHECKED BY: 1/R/W T

SCALE: AS SHOWN

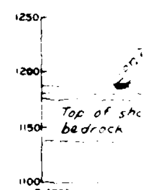
2

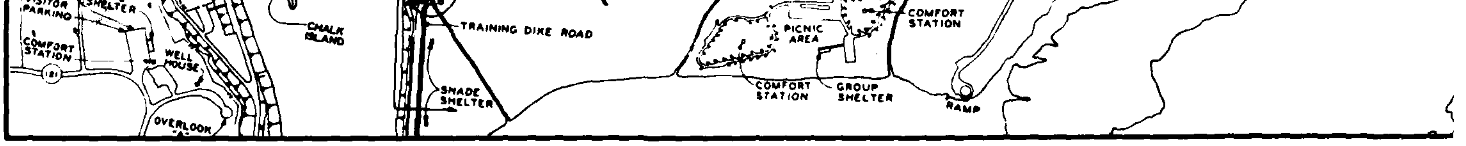


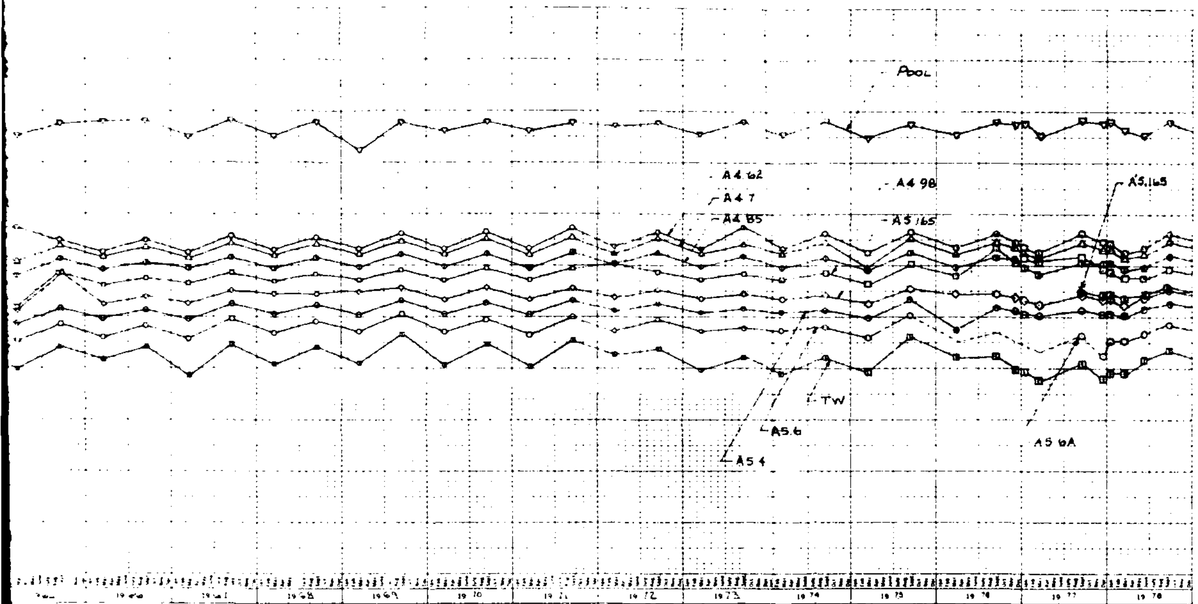


PIEZO

PIEZ NO	DATE
[illegible]	[illegible]
A [illegible]	
A [illegible]	
A 4.[illegible]	
A [illegible]	
A [illegible]	
A [illegible]	"

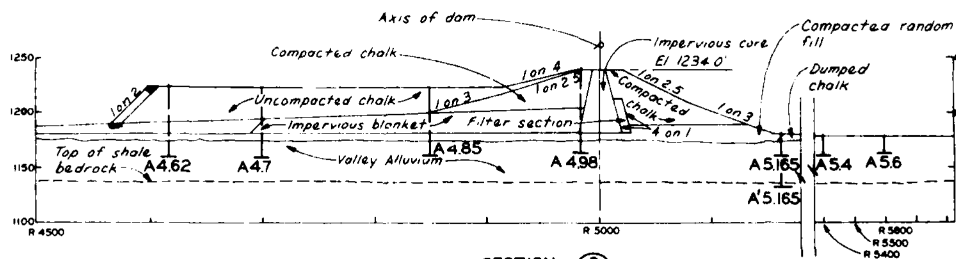






PIEZOMETER EFFECTIVNESS TESTS

PIEZ. NO.	DATE	RESULTS	ACTION
A 4.62	Aug 19-20 1976	OK	–
A 4.7	"	OK	–
A 4.85	"	Negligible response	Rehab. Nov. 76
A 4.98	"	OK	–
A 5.165	"	OK	–
A 5.4	"	OK	–
A 5.6	"	Replaced	A 5.6 A



SECTION 2
NO SCALE

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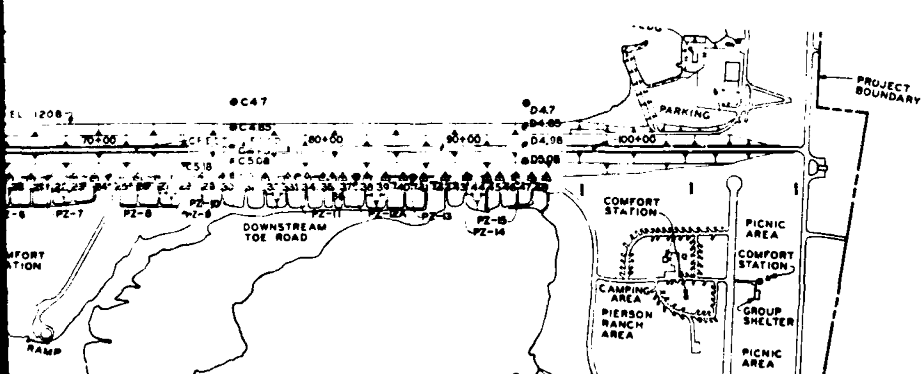


DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

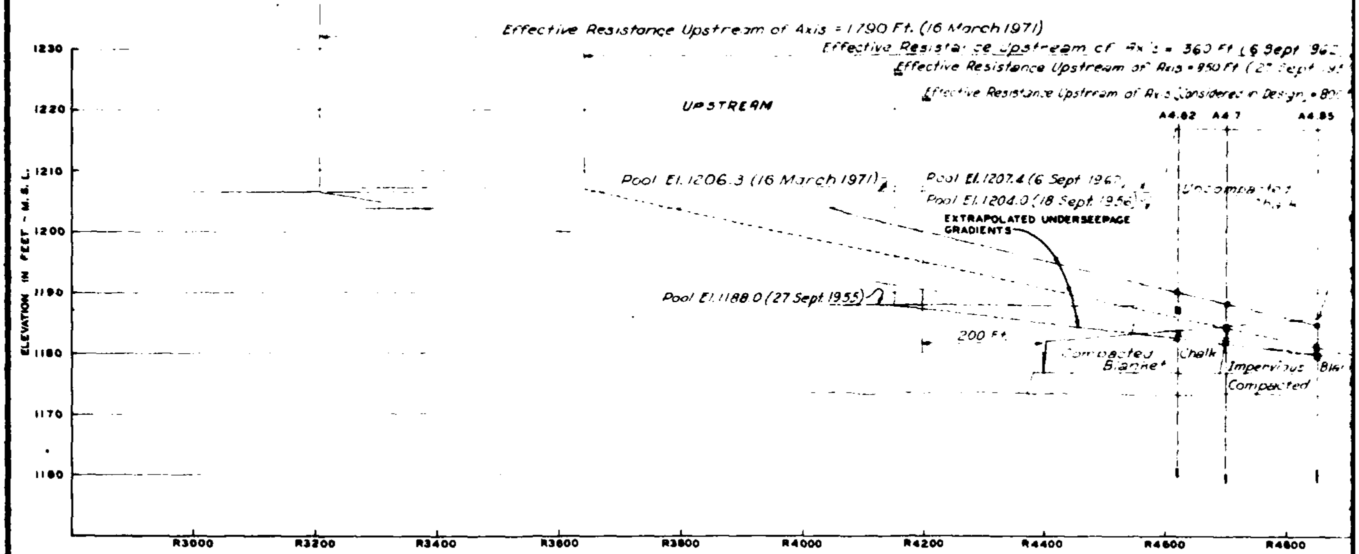
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT DAM - LEWIS AND CLARK LAKE
EMBANKMENT
UNDERSEEPAGE
LINE A - PIEZOMETER OBSERVATIONS

SCALE: AS SHOWN

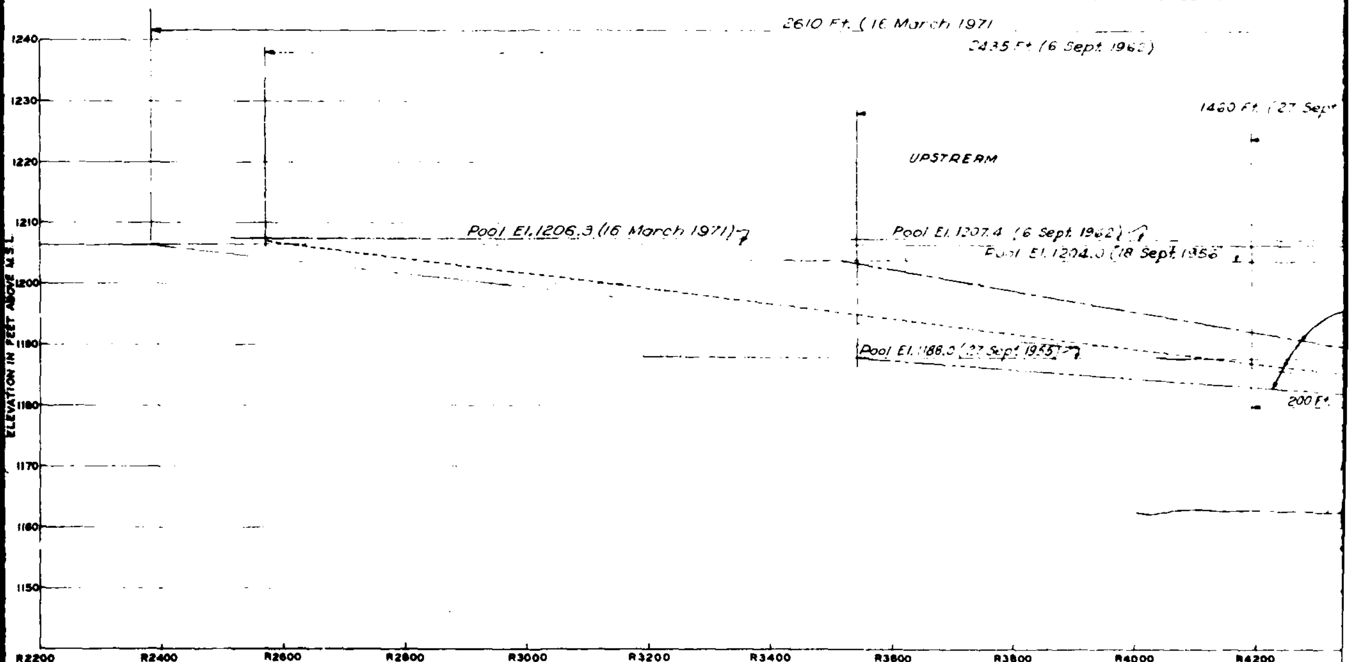
EMBANKMENT CRITERIA AND PERFORMANCE REPORT PLATE A-34

2



PIEZOMETER LINE "A" STA. 27+00

SECTION ADJACENT TO SPILLWAY

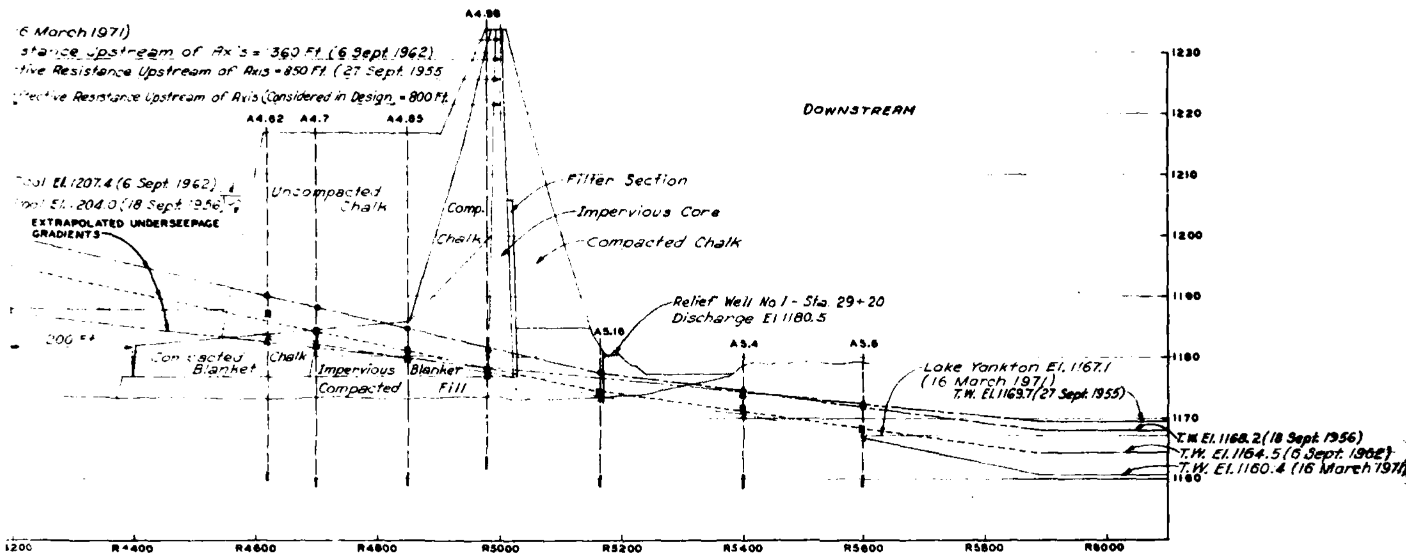


PIEZOMETER LINE "B"

CHUTE SECTION

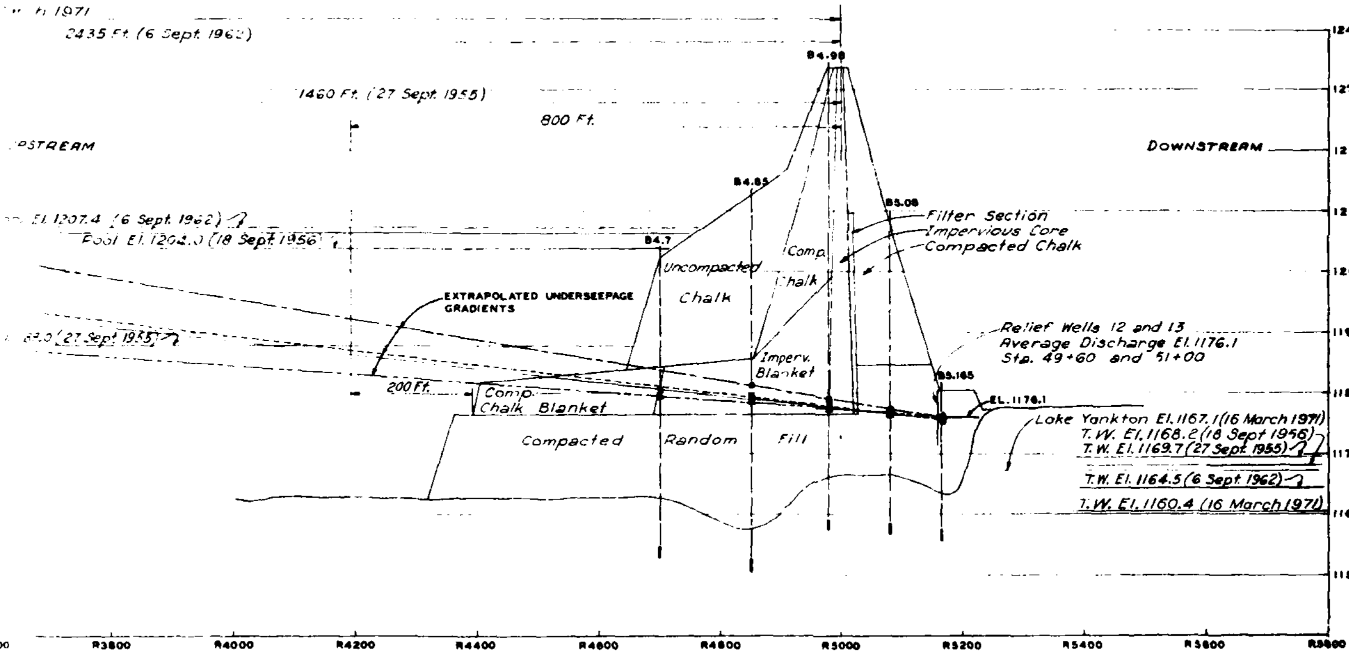
LEGEND:

- ▲ PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (27 SEP
- ● PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (18 SEP
- ■ PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (6 SEP
- x PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (16 MAR



PIEZOMETER LINE "A" STA. 27+00

SECTION ADJACENT TO SPILLWAY



PIEZOMETER LINE "B" STA. 50+00

CHUTE SECTION

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

SCALE: HORIZ. 1 INCH = 100 FEET; VERT. 1 INCH = 10 FEET

LEGEND:

- ▲ PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (27 SEPT. 1955)
- ● PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (18 SEPT 1956)
- ■ PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (6 SEPT 1962)
- x PIEZOMETRIC PRESSURE IN VALLEY ALLUVIUM BENEATH THE DAM (16 MARCH 1971)

U. S. ARMY ENGINEER DISTRICT, OMAHA

MISSOURI RIVER

GAVINS POINT DAM-LEWIS AND CLARK LAKE

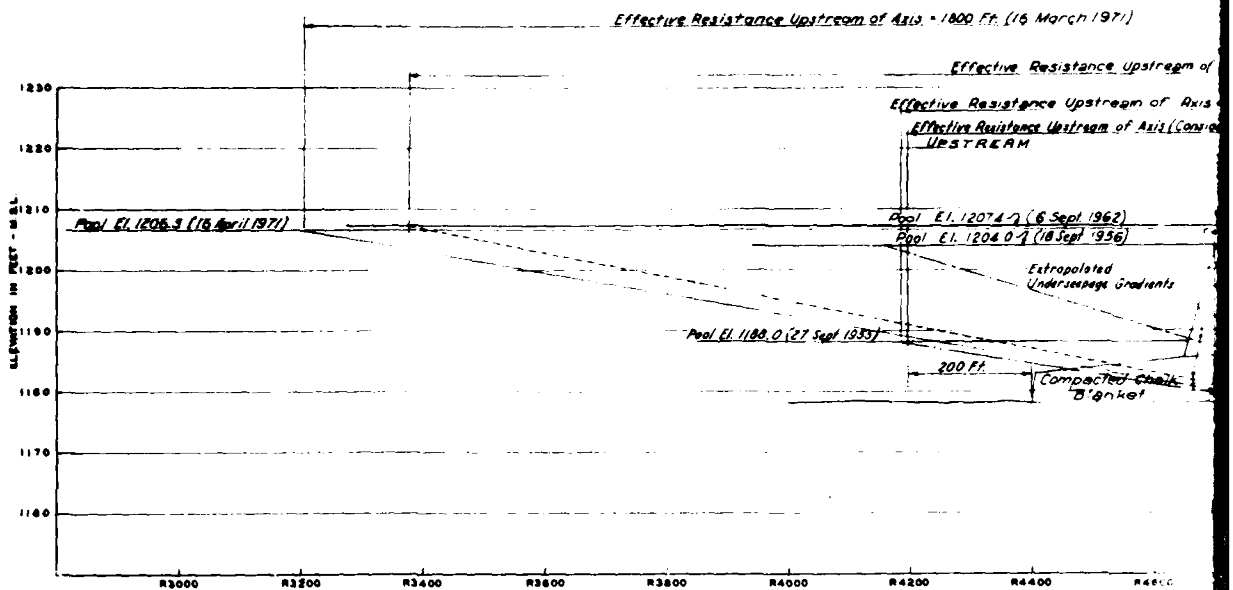
UNDERSEEPAGE

PIEZOMETER LINES A AND B

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

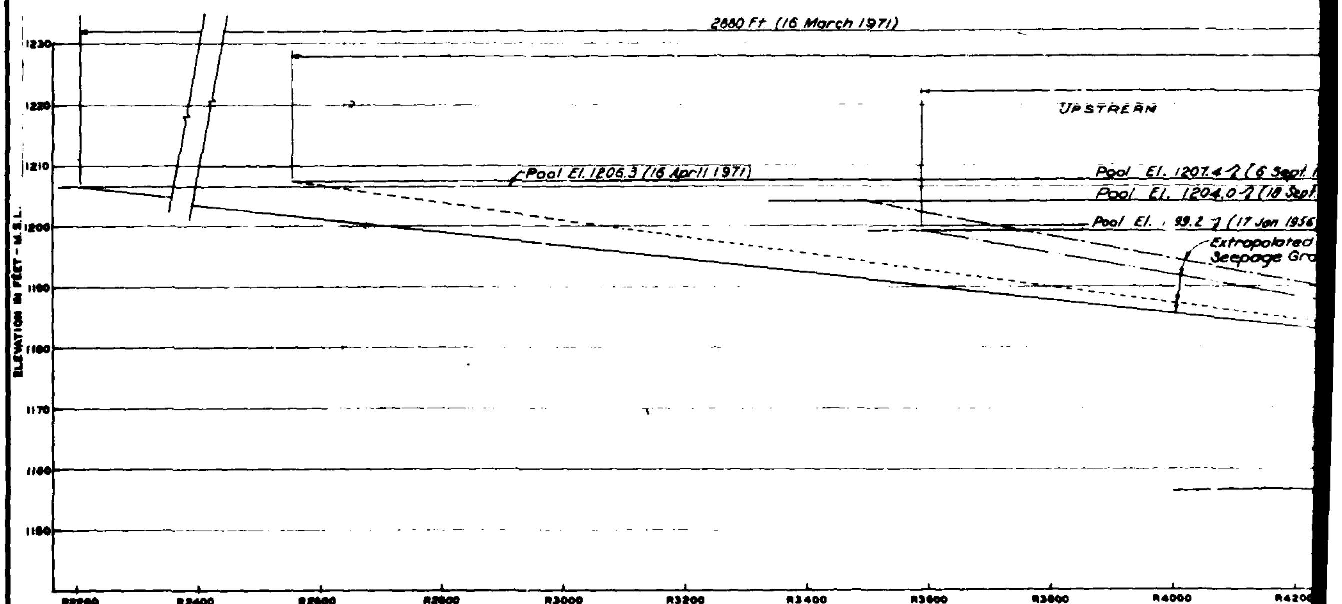
PLATE A-35

2

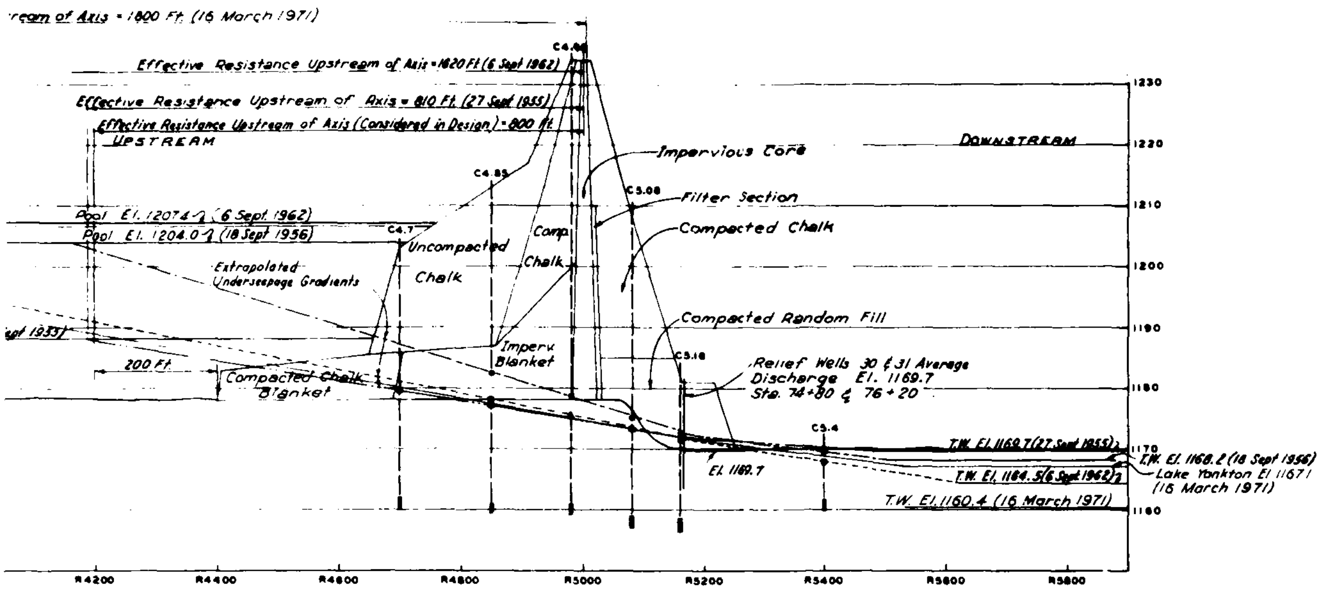


PIEZOMETER LINE "C" ST

ISLAND SECTION

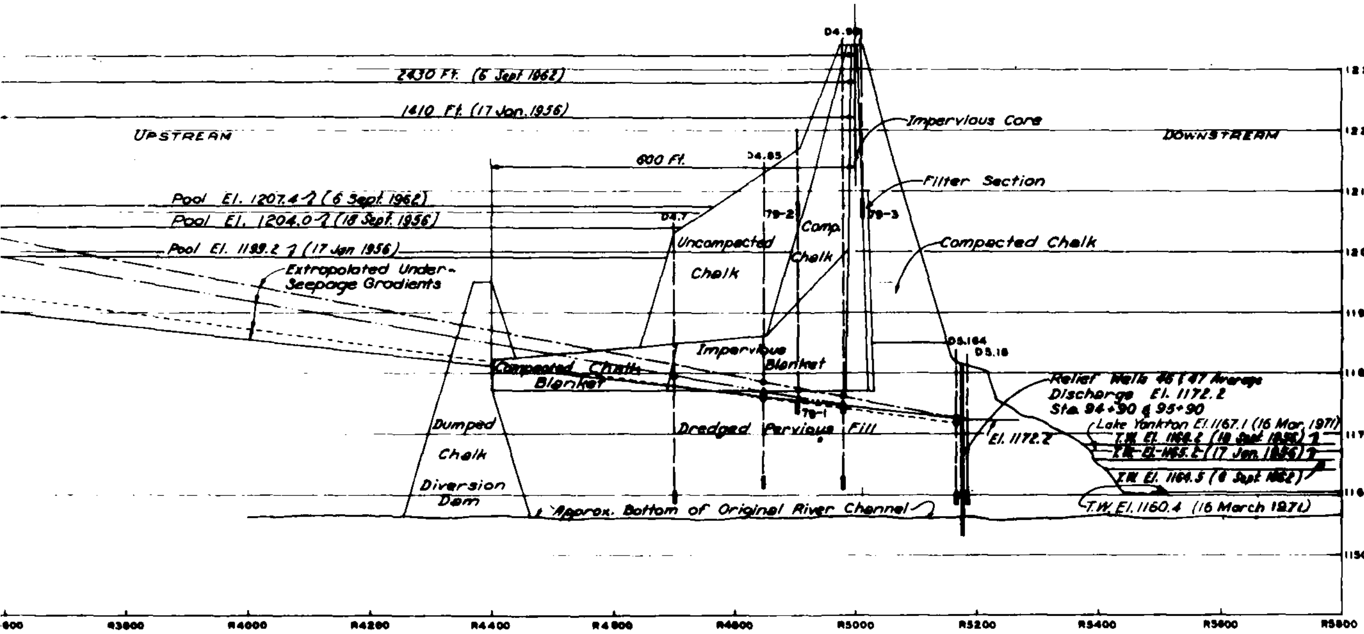


PIEZ



PIEZOMETER LINE "C" STA. 75+00

ISLAND SECTION



PIEZOMETER LINE "D" STA. 95+00

CLOSURE SECTION

NOTE:
FOR LEGEND, SEE PLATE A-35

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

SCALE: VERT. 1 INCH = 10 FEET
HORIZ. 1 INCH = 100 FEET

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

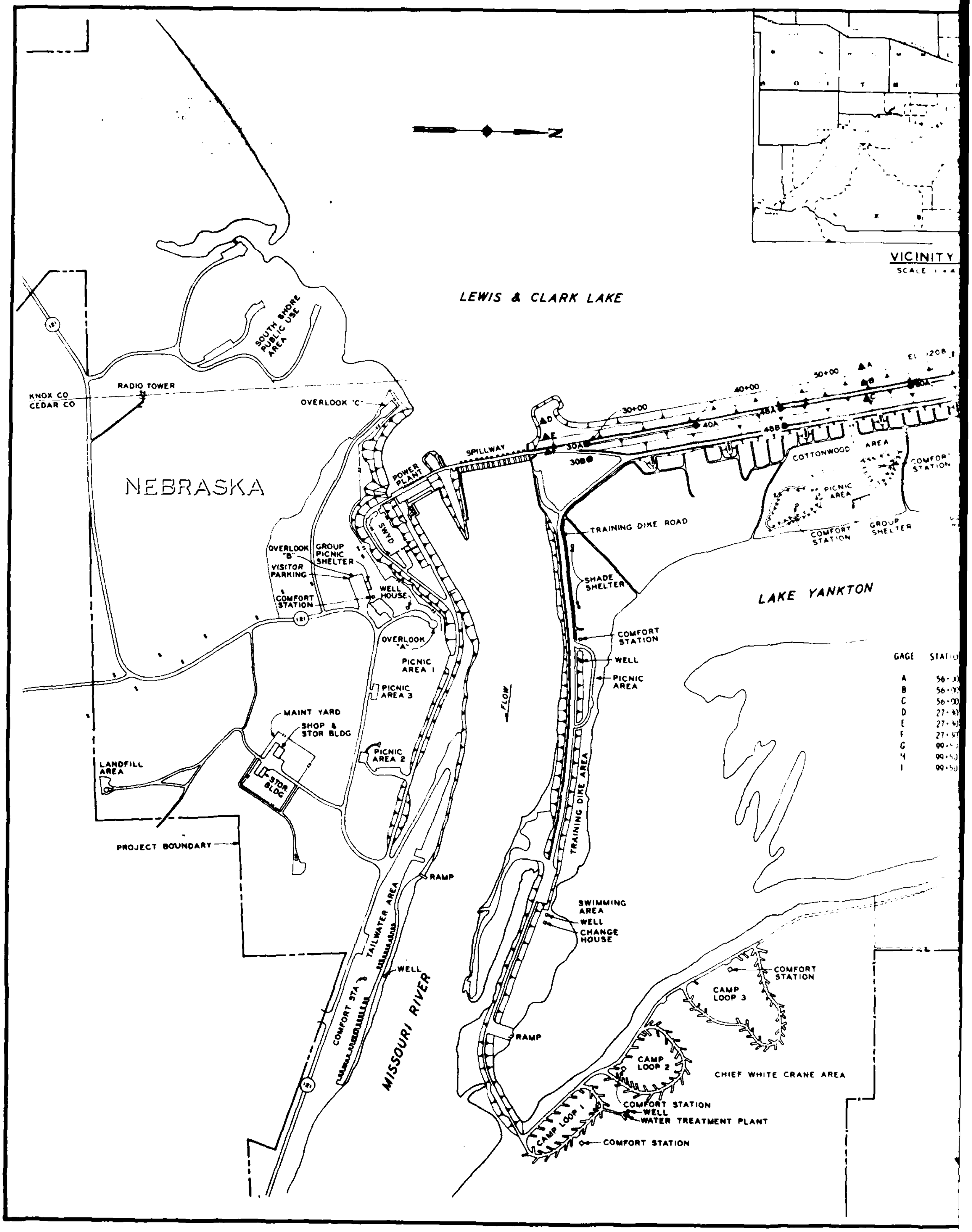
DATE	DESCRIPTION	MADE	APPR'D

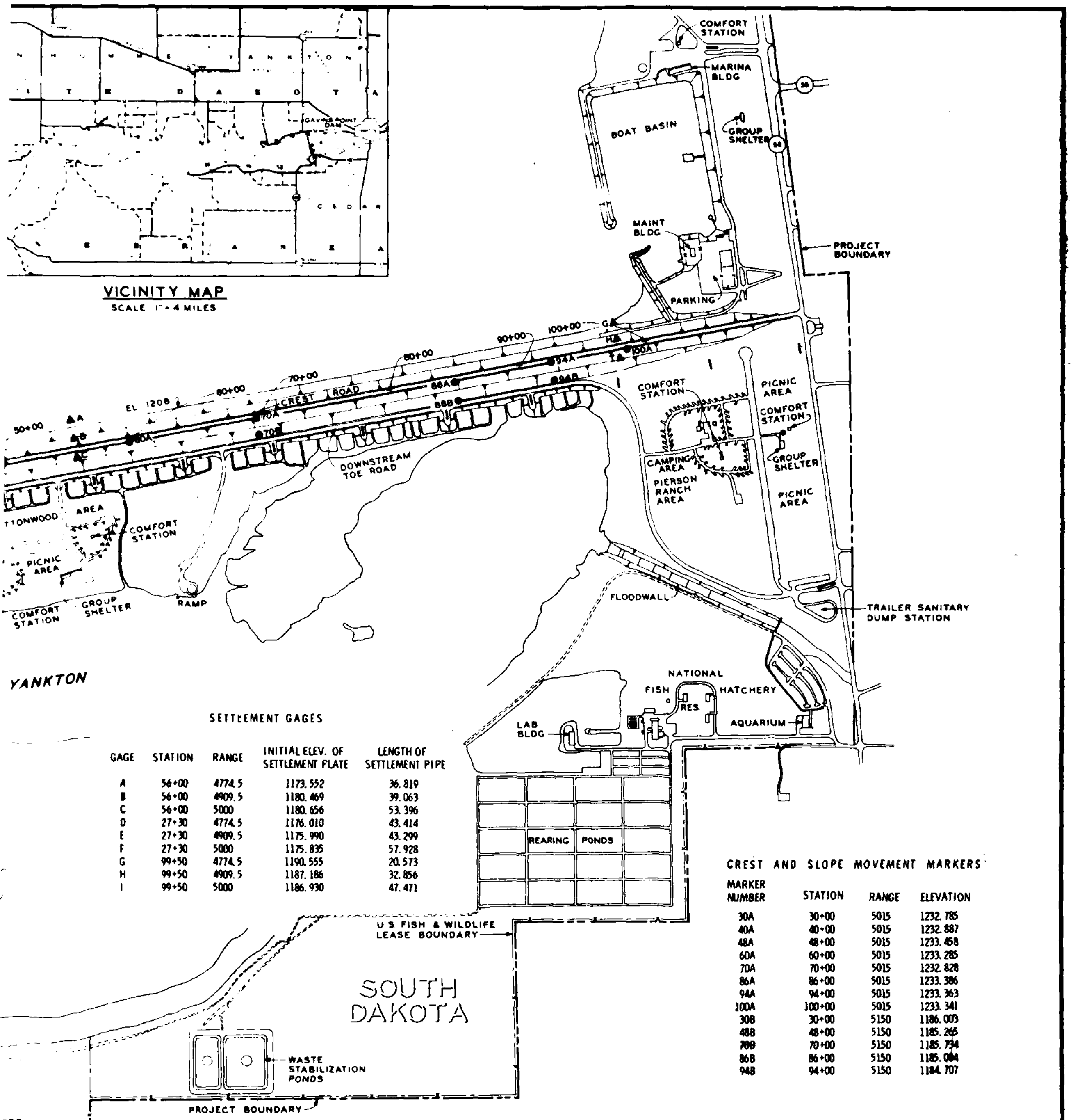
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT DAM-LEWIS AND CLARK LAKE
UNDERSEEPAGE
PIEZOMETER LINES C AND D

DRAWN BY: L M G
CHECKED BY: E G H
DATE: MAY 1972
SCALE: AS SHOWN


LEWIS & CLARK LAKE
NEBRASKA
SOUTH SHORE PUBLIC USE AREA
RADIO TOWER
KNOX CO
CEDAR CO
OVERLOOK "C"
SPILLWAY
POWER PLANT
30+00
40+00
50+00
TRAINING DIKE ROAD
COTTONWOOD AREA
COMFORT STATION
PICNIC AREA
COMFORT STATION
GROUP SHELTER
OVERLOOK "B"
GROUP PICNIC SHELTER
VISITOR PARKING
COMFORT STATION
WELL HOUSE
SHADE SHELTER
LAKE YANKTON
OVERLOOK "A"
COMFORT STATION
WELL
PICNIC AREA 1
PICNIC AREA
PICNIC AREA 3
FLOW
MAINT YARD
SHOP & STOR BLDG
STOR BLDG
PICNIC AREA 2
LANDFILL AREA
PROJECT BOUNDARY
TRAINING DIKE AREA
RAMP
TAILWATER AREA
SWIMMING AREA
WELL
CHANGE HOUSE
WELL
COMFORT STA
MISSOURI RIVER
COMFORT STATION
CAMP LOOP 3
RAMP
CAMP LOOP 2
CHIEF WHITE CRANE AREA
COMFORT STATION
WELL
WATER TREATMENT PLANT
CAMP LOOP 1
COMFORT STATION
VICINITY
GAGE
A
B
C
D
E
F
G
H
I



SETTLEMENT GAGES

GAGE	STATION	RANGE	INITIAL ELEV. OF SETTLEMENT PLATE	LENGTH OF SETTLEMENT PIPE
A	56+00	4774.5	1173.552	36.819
B	56+00	4909.5	1180.469	39.063
C	56+00	5000	1180.656	53.396
D	27+30	4774.5	1176.010	43.414
E	27+30	4909.5	1175.990	43.299
F	27+30	5000	1175.835	57.928
G	99+50	4774.5	1190.555	20.573
H	99+50	4909.5	1187.186	32.856
I	99+50	5000	1186.930	47.471

CREST AND SLOPE MOVEMENT MARKERS

MARKER NUMBER	STATION	RANGE	ELEVATION
30A	30+00	5015	1232.785
40A	40+00	5015	1232.887
48A	48+00	5015	1233.458
60A	60+00	5015	1233.285
70A	70+00	5015	1232.828
86A	86+00	5015	1233.386
94A	94+00	5015	1233.363
100A	100+00	5015	1233.341
30B	30+00	5150	1186.003
48B	48+00	5150	1185.265
70B	70+00	5150	1185.734
86B	86+00	5150	1185.004
94B	94+00	5150	1184.707

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

SCALE IN FEET
400 0 400 800 1200

SAFETY PAYS

DATE	DESCRIPTION	MADE	APP'D
6 72	Revised drawing number & misc revisions	HDG	RLB

REVISIONS

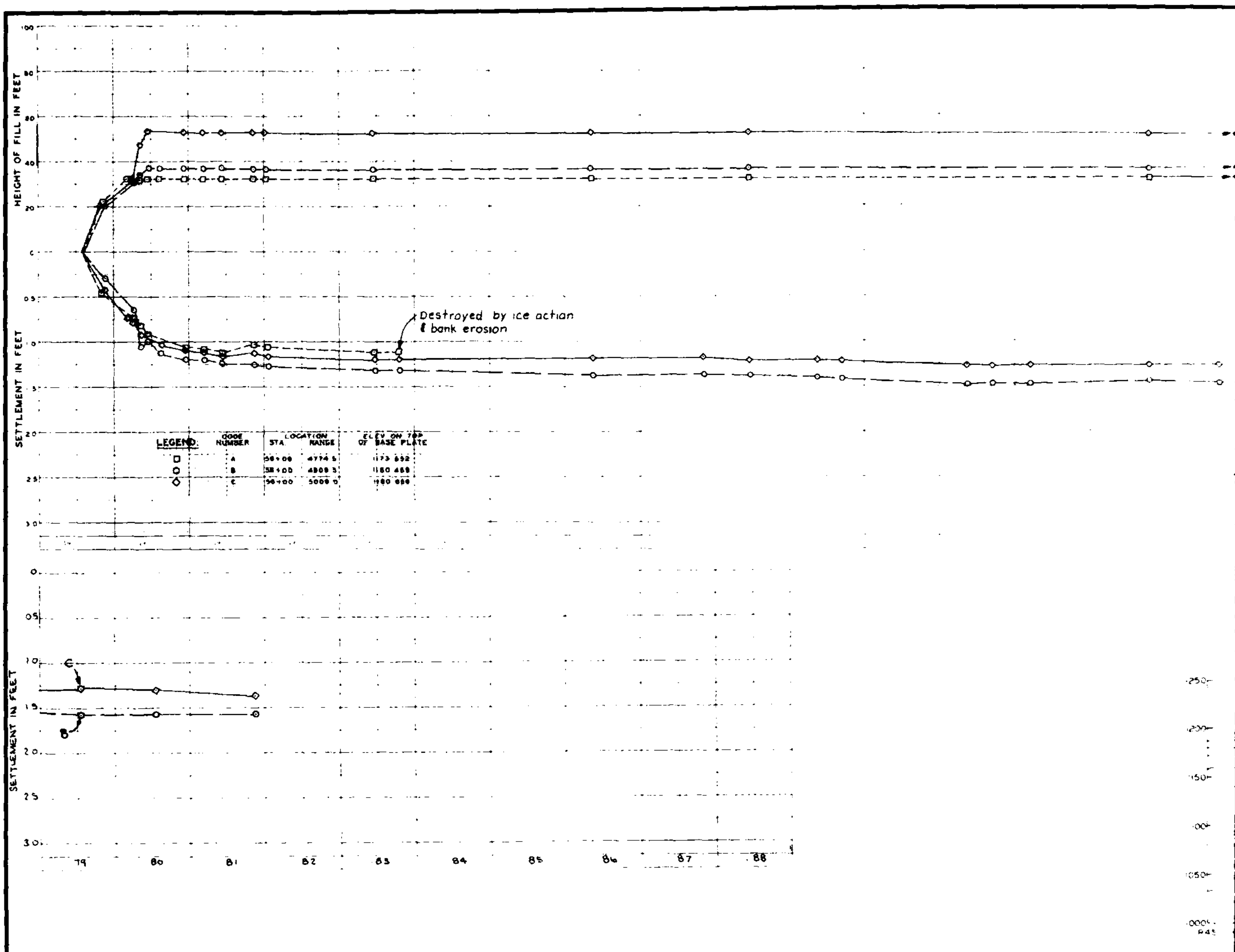
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: EON
DRAWN BY: MBH
CHECKED BY: EON

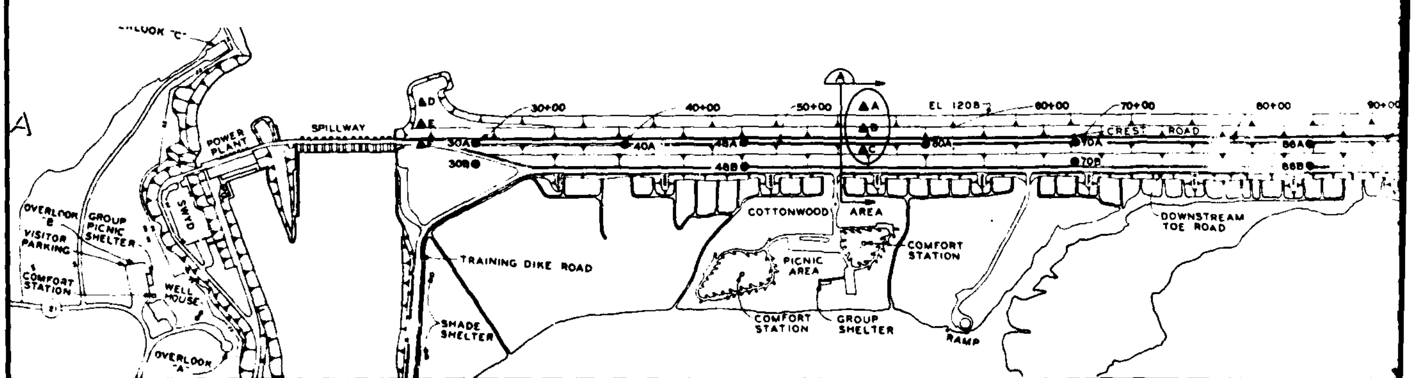
MISSOURI RIVER
GAVINS POINT DAM-LEWIS AND CLARK LAKE
EMB. AND FOUNDATION SETTLEMENT LOCATION PLAN
EMBANKMENT SECTIONS, SETTLEMENT GAGES AND CREST AND SLOPE MOVEMENT MARKERS

DATE: MAY 1972
SCALE: AS SHOWN
DRAWING NUMBER: MG-OPN-31 E1.1

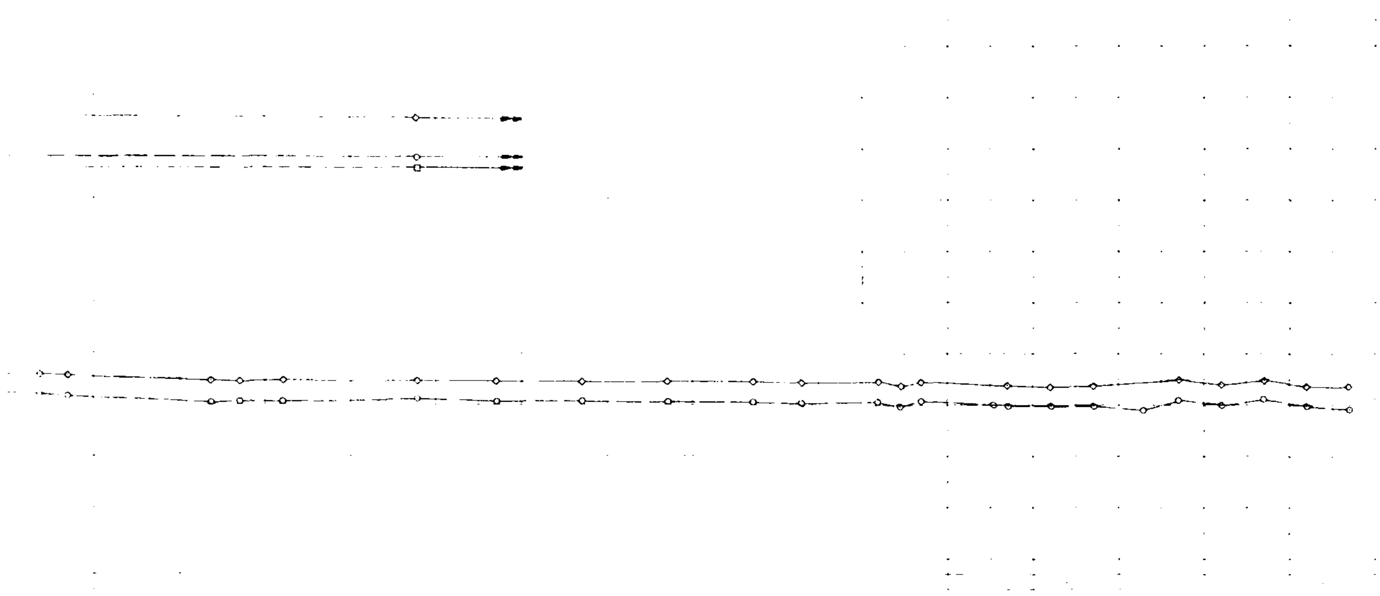
THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

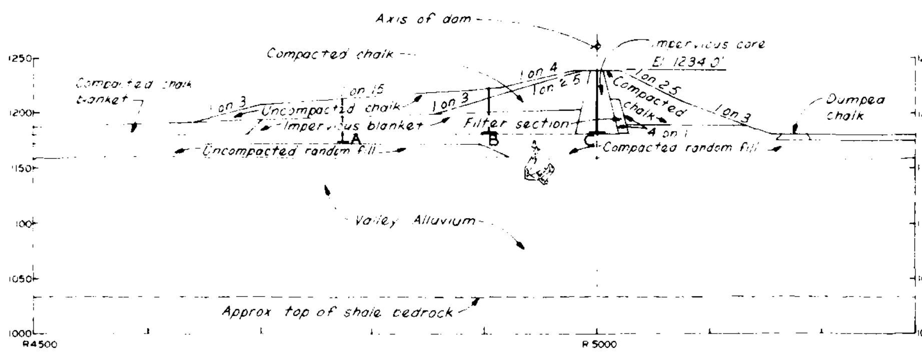


HEIGHT OF FILL IN FEET
SETTLEMENT IN FEET
Destroyed by ice action
& bank erosion
LEGEND:
CODE NUMBER
LOCATION
STA
RANGE
ELEV ON TOP OF BASE PLATE
A
B
C
79
80
81
82
83
84
85
86
87
88



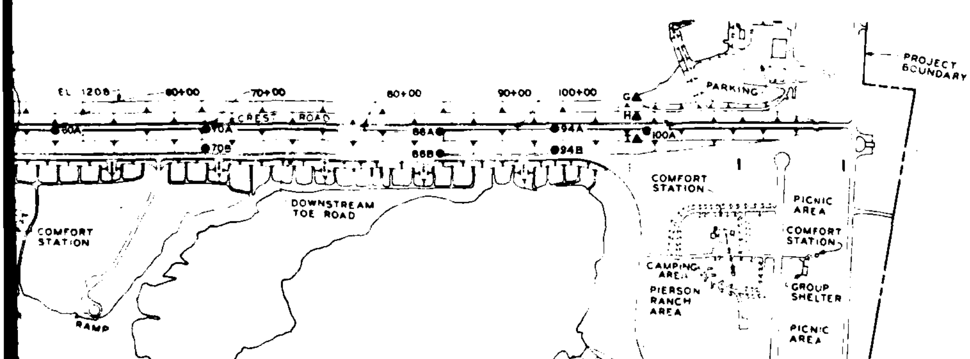
POWER PLANT
SPILLWAY
TRAINING DIKE ROAD
SHADE SHELTER
30+00
40+00
50+00
60+00
70+00
80+00
90+00
EL 1208
CREST ROAD
COTTONWOOD
AREA
COMFORT STATION
PICNIC AREA
GROUP SHELTER
RAMP
DOWNSTREAM TOE ROAD
OVERLOOK B
GROUP PICNIC SHELTER
VISITOR PARKING
COMFORT STATION
WELL HOUSE
OVERLOOK A





SECTION (A)
NO SCALE

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE



DATE	DESCRIPTION	MADE	APPR'D

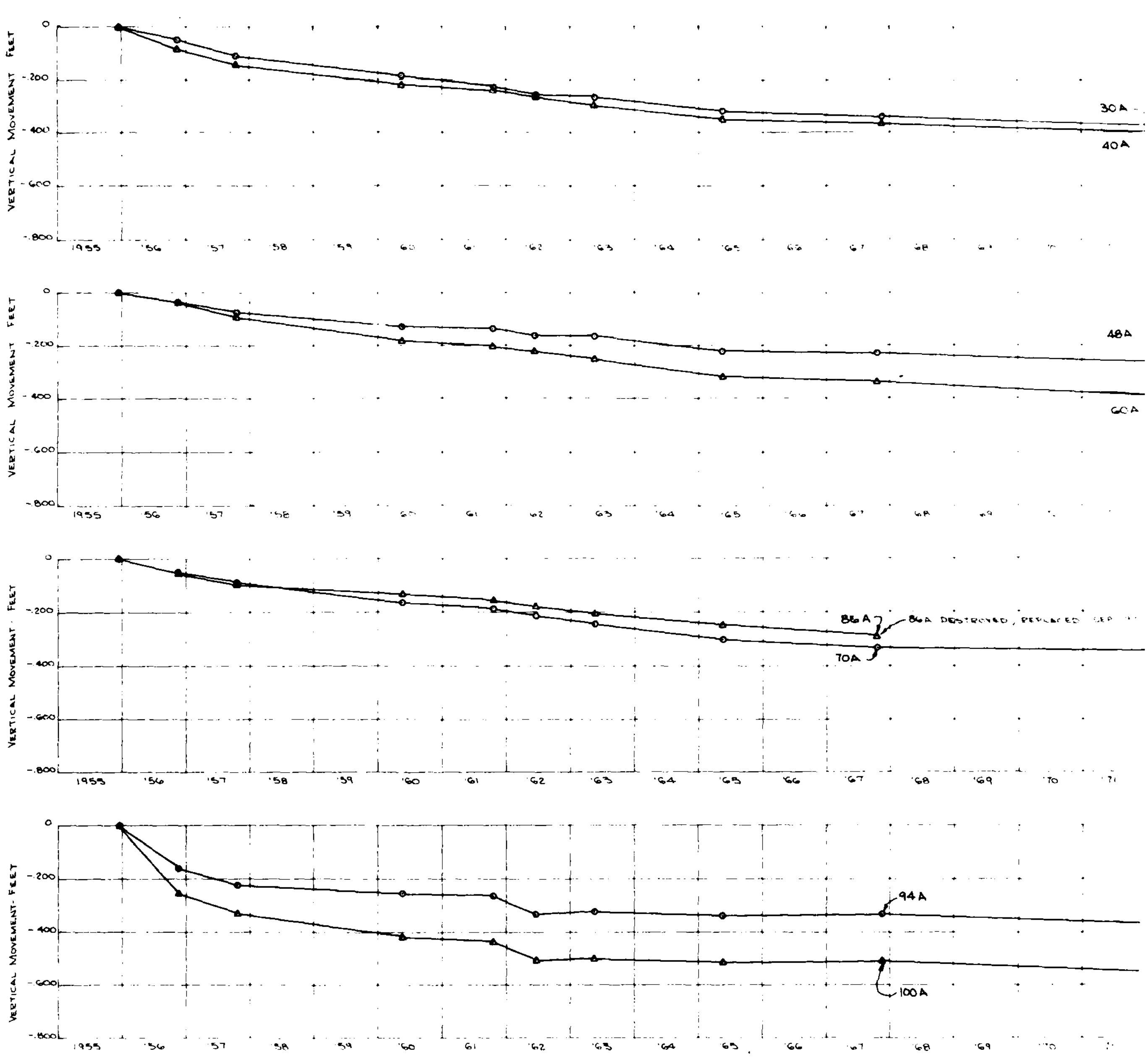
REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

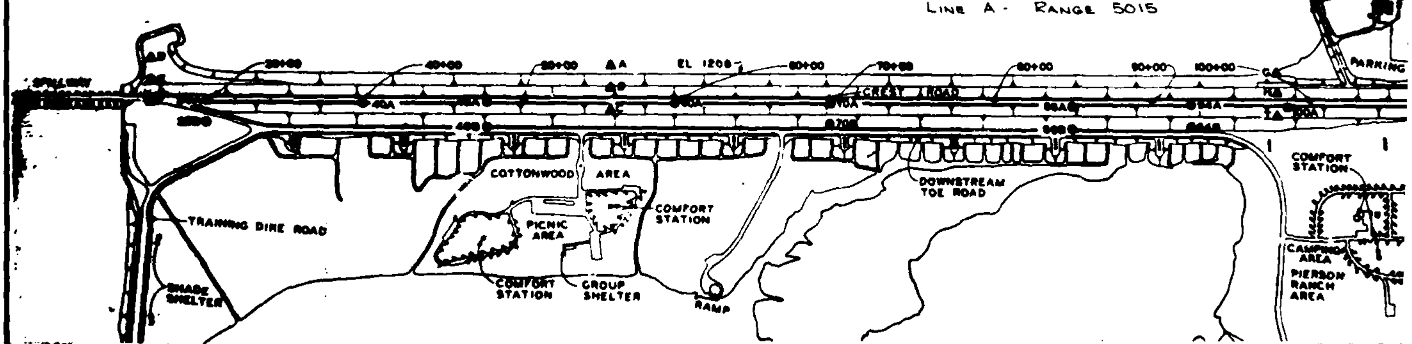
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
CHIEF SECTION
RECOMMENDED:
CHIEF BRANCH
APPROVED:

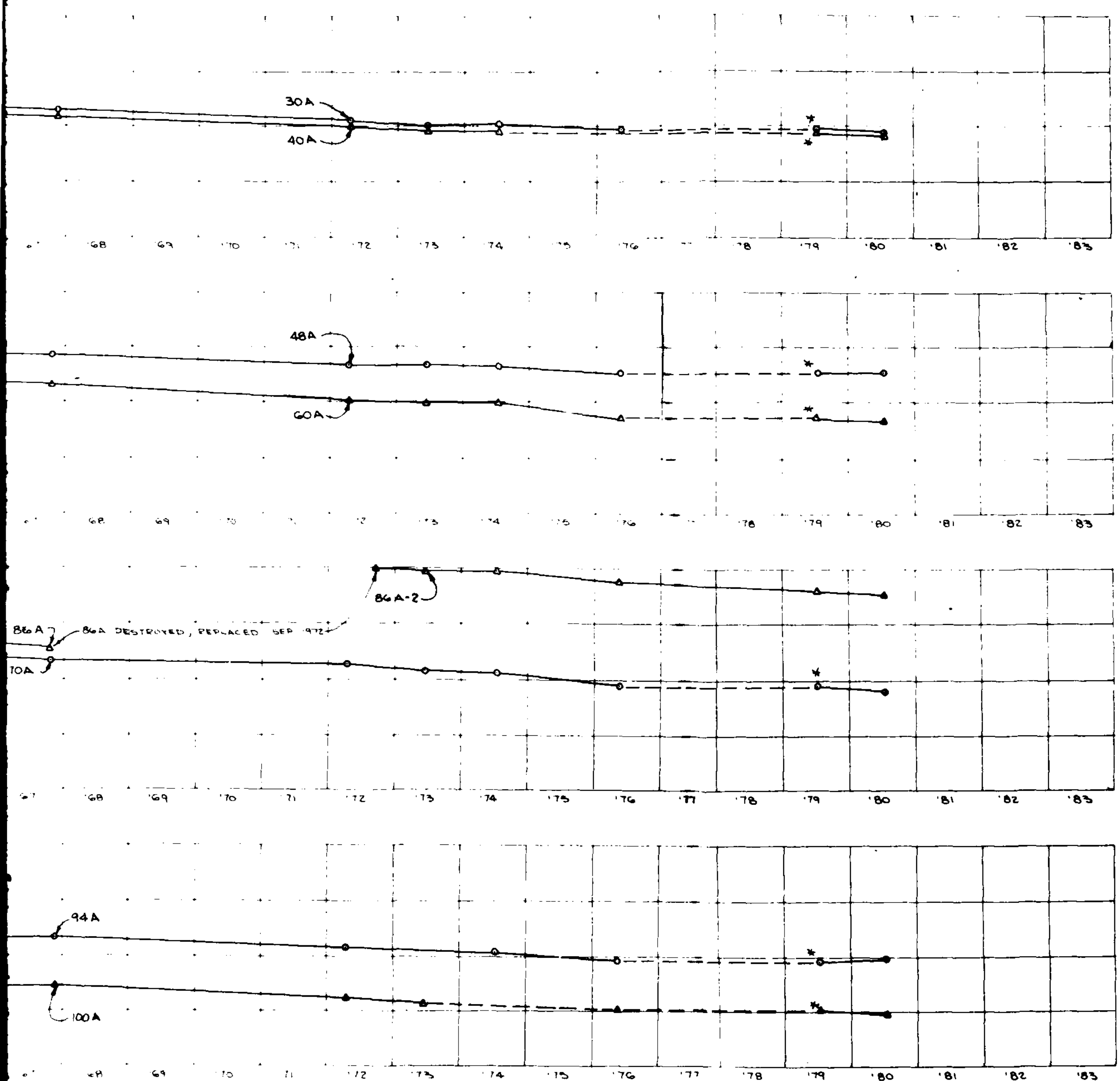
MISSOURI RIVER
GAVINS POINT DAM - LEWIS AND CLARK LAKE
EMBANKMENT
SETTLEMENT GAGE OBSERVATIONS
GAGES A, B AND C

APPROVED: CHIEF ENGINEERING DIVISION DATE
SCALE AS SHOWN SPEC. NO.
DRAWING NUMBER
COL. C. E. DISTRICT ENGINEER SHEET



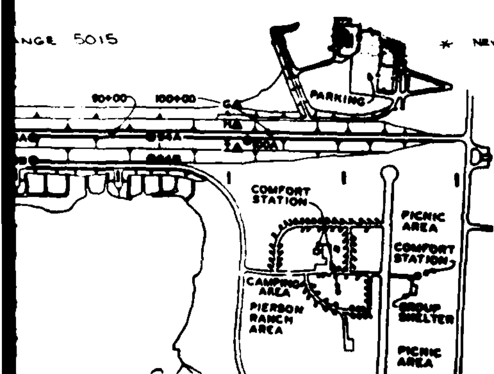
LINE A - RANGE 5015





ANGE 5015

* NEW MARKERS INSTALLED JUL '79



THIS DRAWING [redacted] TO THREE-EIGHTHS THE [redacted] SCALE.

SAFETY PAYS

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

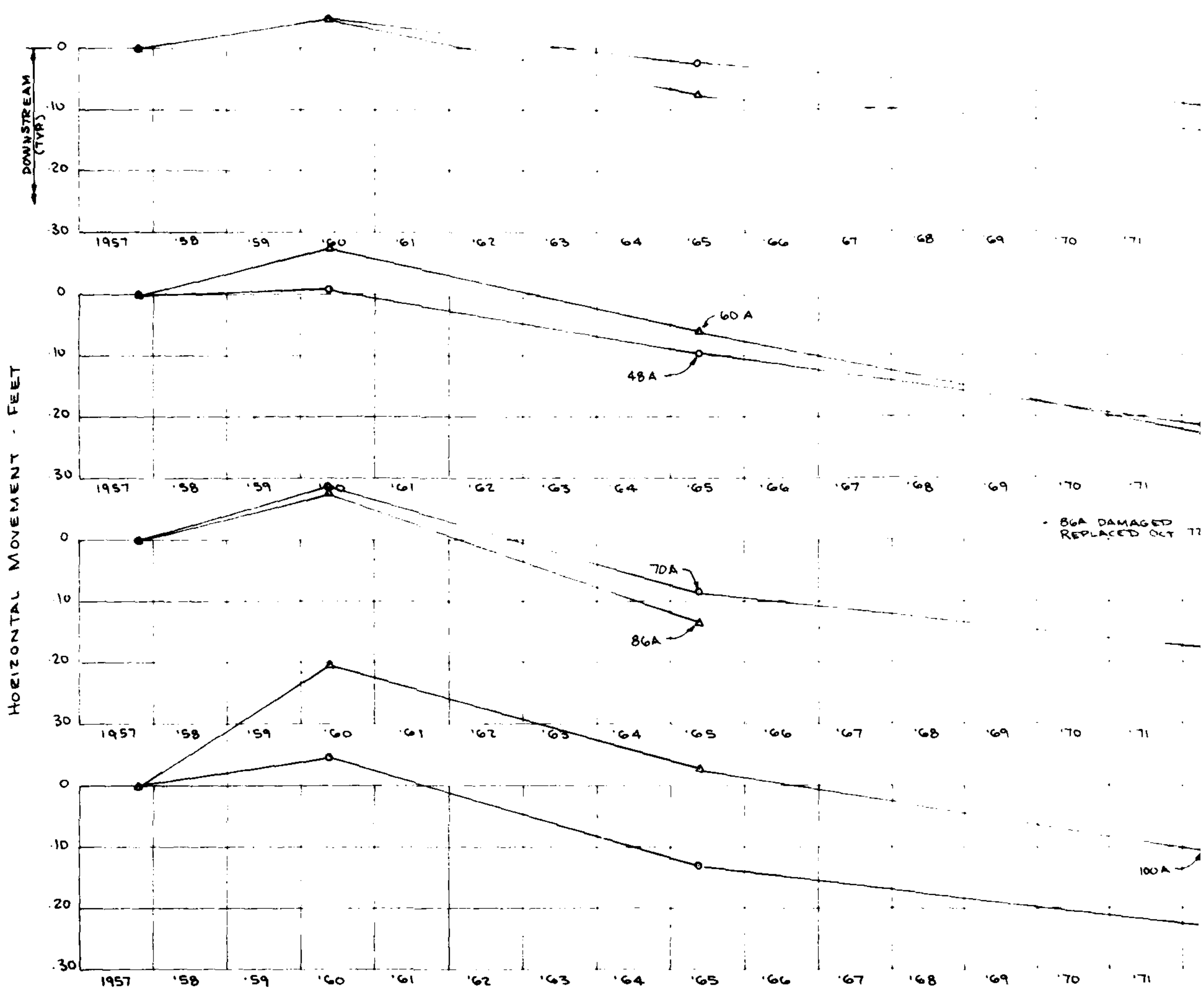
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

CHECKED BY: 1/20/81 TJM

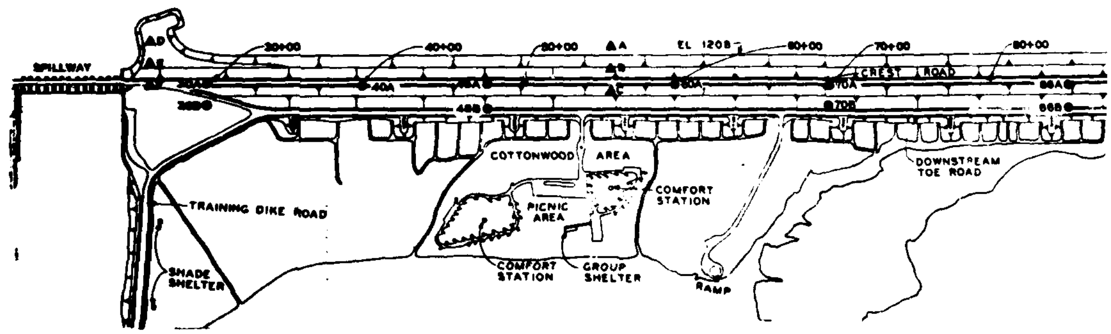
MISSOURI RIVER
GAVINS POINT DAM - LEWIS & CLARK LAKE
EMBANKMENT & FOUNDATION SETTLEMENT
CREST MOVEMENT MARKERS
VERTICAL MOVEMENT — HISTORIES
LINE A - RANGE 5015

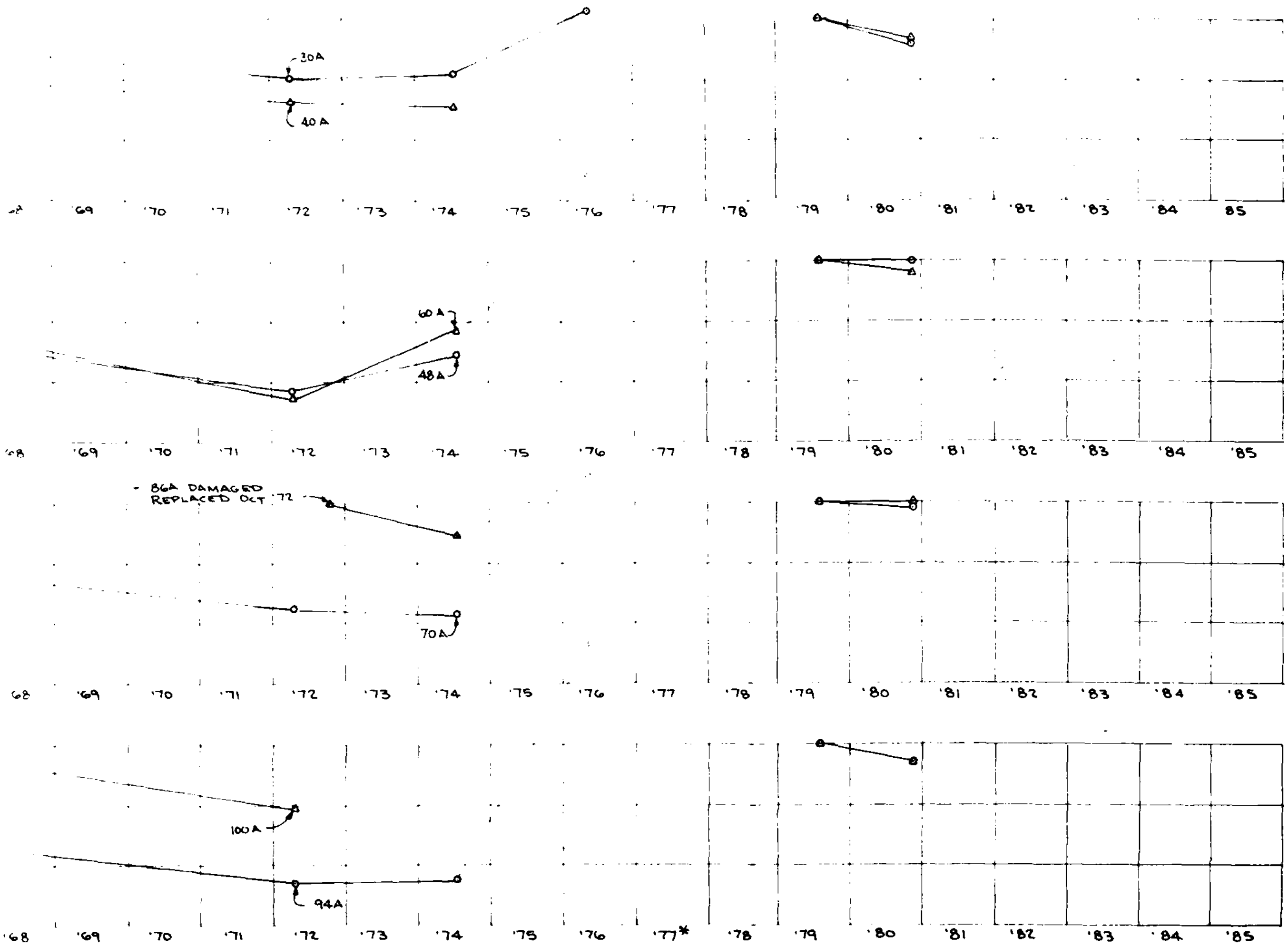
SCALE AS SHOWN

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.



LINE A - RANGE 5015

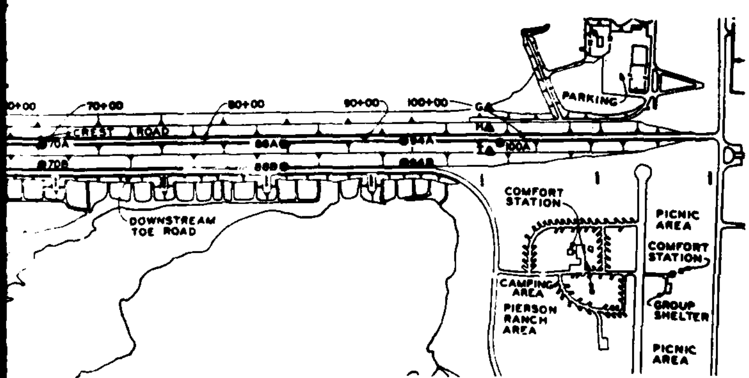




LINE "A" - RANGE 5015

* NEW MARKERS INSTALLED JULY '79 NOT SURVEYED

NOTE: GEODOLITE SURVEYS BEGAN JULY '79



THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
GAVINS POINT DAM
EMBANKMENT
CREST MOVEMENT MARKERS
LINE "A" RANGE 5015
HORIZONTAL MOVEMENT- HISTORIES

CHECKED BY: 1/20/81 TM

SCALE AS SHOWN

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

APPENDIX B
PHOTOS

Photo No. 1 - Aerial view of Gavins Point Dam, looking north. Sept 1978.

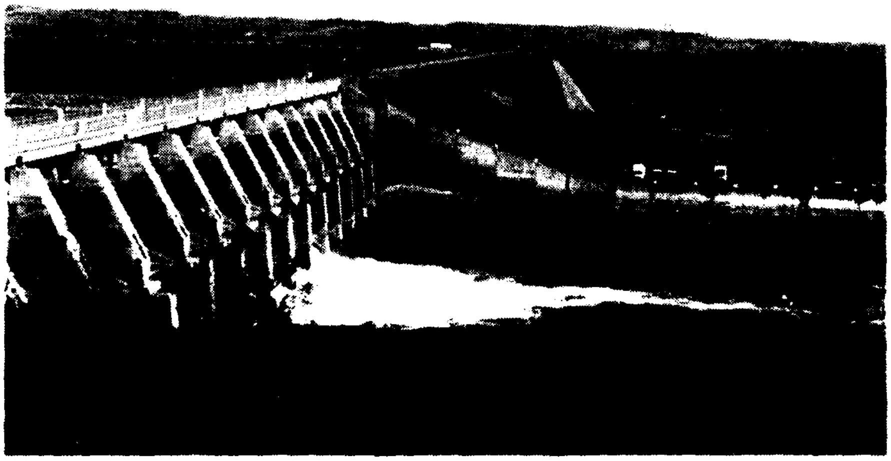


Photo No. 2 - Spillway and embankment, looking north. May 1979.

Plate 1

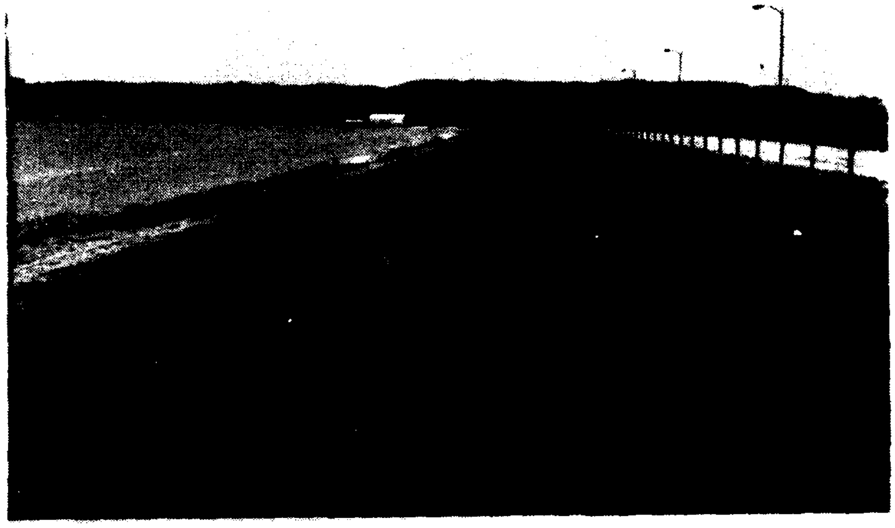


Photo No. 3 - Upstream slope of embankment, looking north. May 1979.

Photo No. 4 - Field boulder upstream slope protection. May 1979.

Photo No. 5 - Embankment downstream slope, toe road, and downstream area. May 1979.

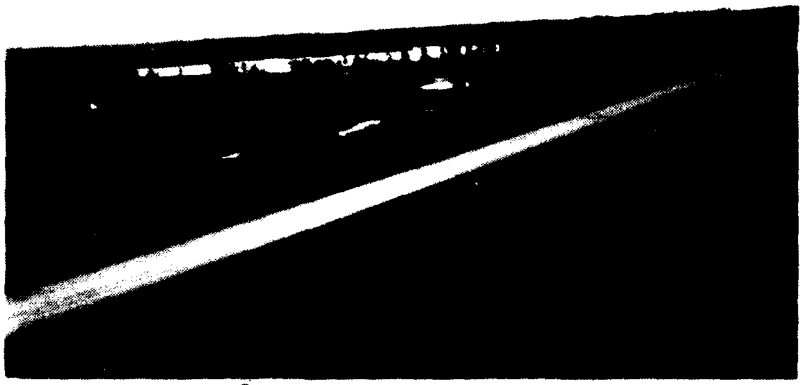


Photo No. 6 - View of downstream area showing toe road, covers of relief well No. 36, piezometer No. 12, and relief well No. 37, and relief well discharge ditches. Oct 1972.

Photo No. 7 - Aerial view of dam site during Earthwork, Stage I construction, showing dike across river chute, looking southeast. Sept 1952.

Photo No. 8 - Aerial view of spur dike closure operations of middle river chute. Sept 1952.

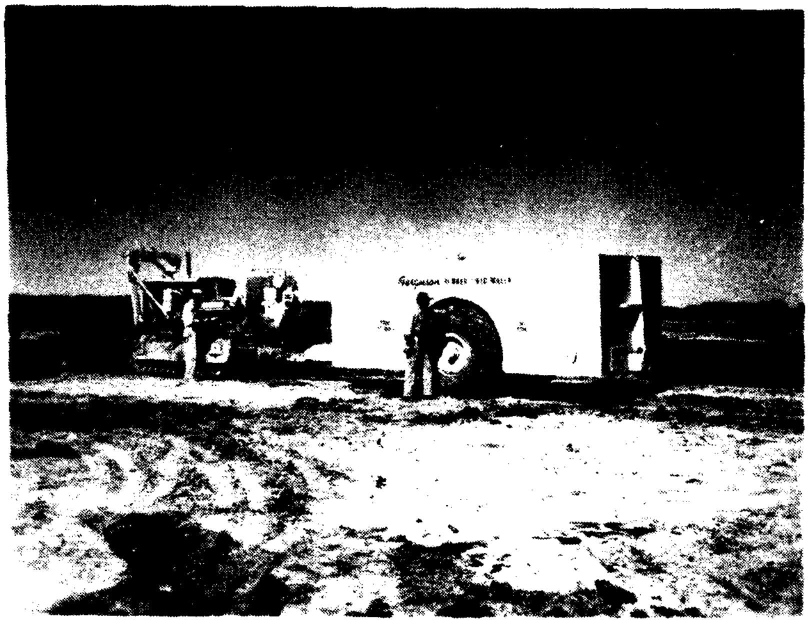


Photo No. 9 - Ferguson 50-ton rubber-tired roller used to compact embankment, Earthwork Stage I. Aug 1952.

Photo No. 10 - Aerial view of embankment construction, Earthwork Stage II, looking north towards closure area and left abutment. Aug 1953.

Photo No. 11 - High altitude aerial photo of damsite during Earthwork Stage II construction. Nov 1953.

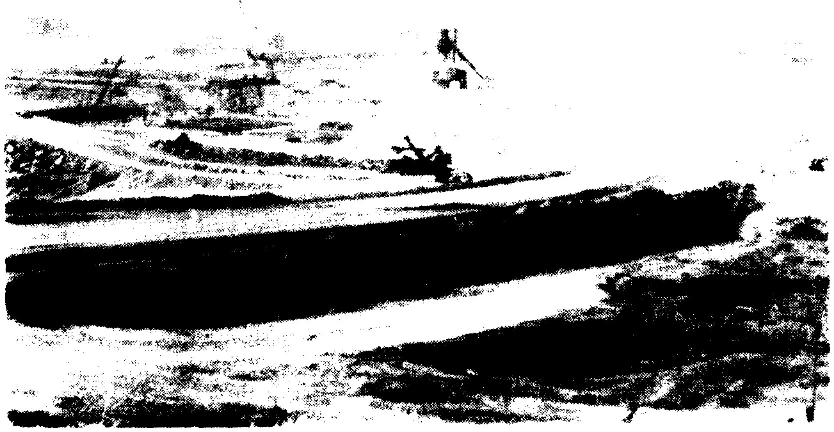


Photo No. 12 - View of construction operations looking northwest. Aug 1953.

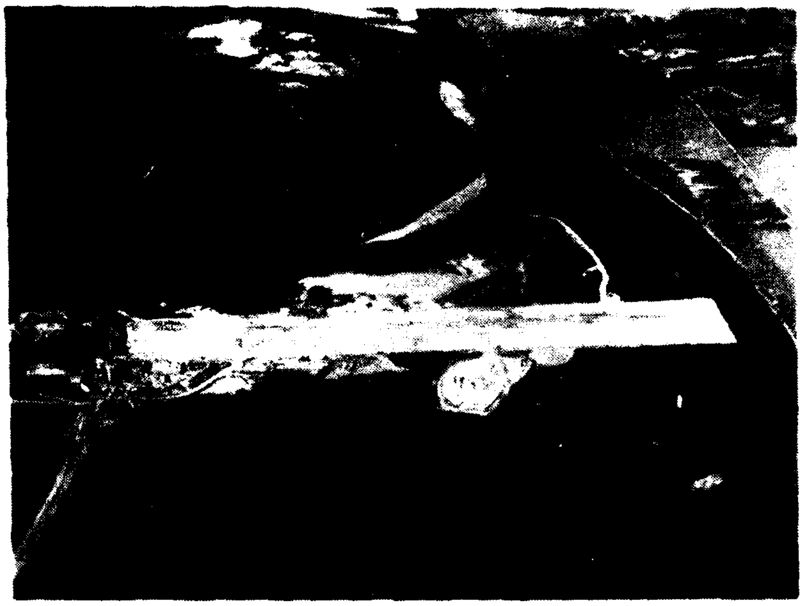


Photo No. 13 - Aerial view of project, looking upstream (west). Oct 1954.

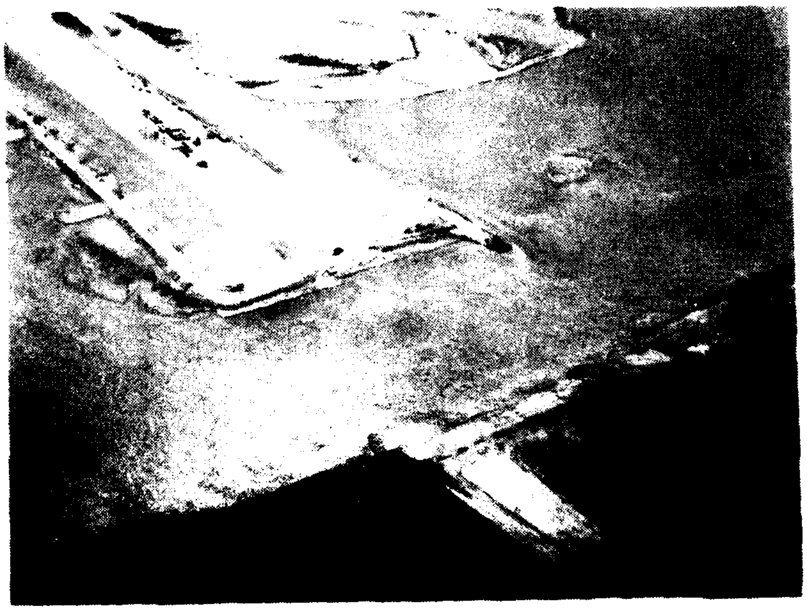


Photo No. 14 - Aerial view of embankment closure area at beginning of closure operations. 29 Jun 1955.

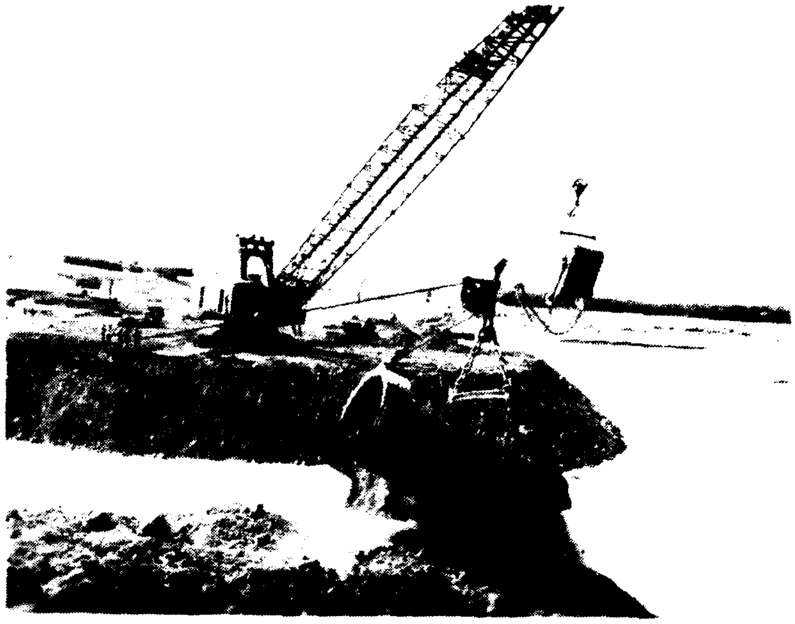


Photo No. 15 - Closure operations, looking south. In foreground is 3.5 C.Y. bucket from 111-M dragline. Opposite bank shows 191-M dragline with 8 C.Y. bucket. 30 Jul 1955.

Photo No. 16 - Closure operations being witnessed by spectators at 10:30 p.m., 30 Jul 1955, about 6 hours before closure.

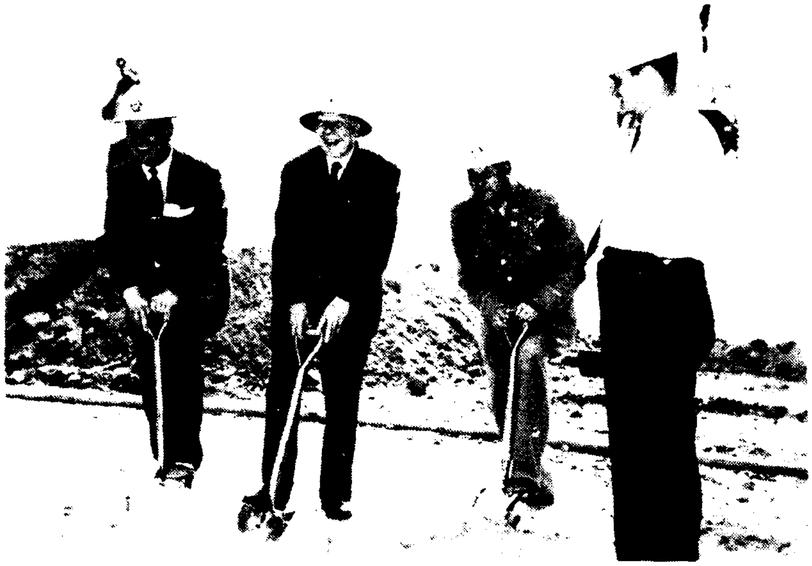


Photo No. 17 - Closure ceremony. Pictured from left are Gov. V. E. Andersen of Nebraska, Secretary of the Army W. M. Brucker, Chief of Engineers Lt. Gen. S. D. Sturgis, and Gov. J. Foss of South Dakota. 31 Jul 1955.

Photo No. 18 - Aerial view of closure section, approximately 6 hours after initial closure was made at about 4:00 a. m., 31 Jul 1955.

Photo No. 19 - Placement of impervious blanket in closure section, looking south. 5 Aug 1955.

Photo No. 20 - Placement of impervious blanket in closure section, looking southwest. 5 Aug 1955.

Photo No. 21 - Aerial view of south abutment area, showing powerhouse and spillway structures, the south portion of embankment, and the diversion channel. Aug 1955 (est.)

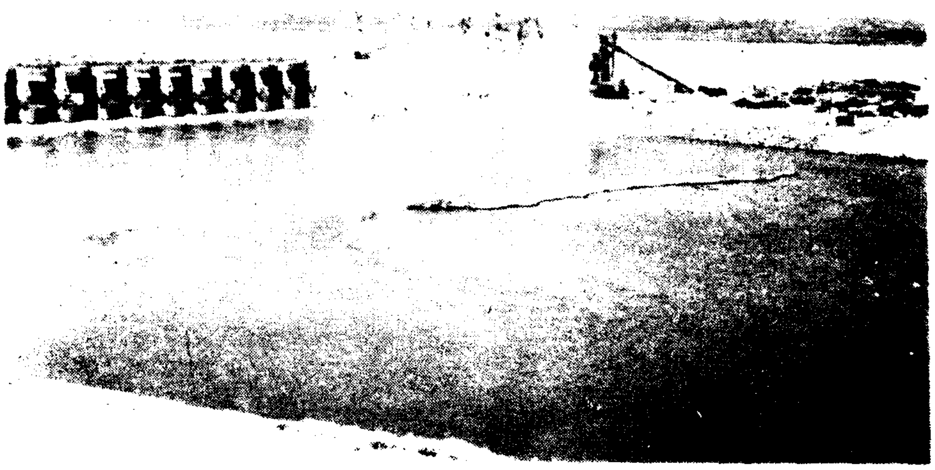


Photo No. 22 - View of the spillway, discharge channel and embankment. Concrete batch plant and contractor's work area shown downstream of the embankment. Aug 1955.

Photo No. 23 - Aerial view of the relief well discharge ditches in embankment closure area, looking south. 8 May 1957.

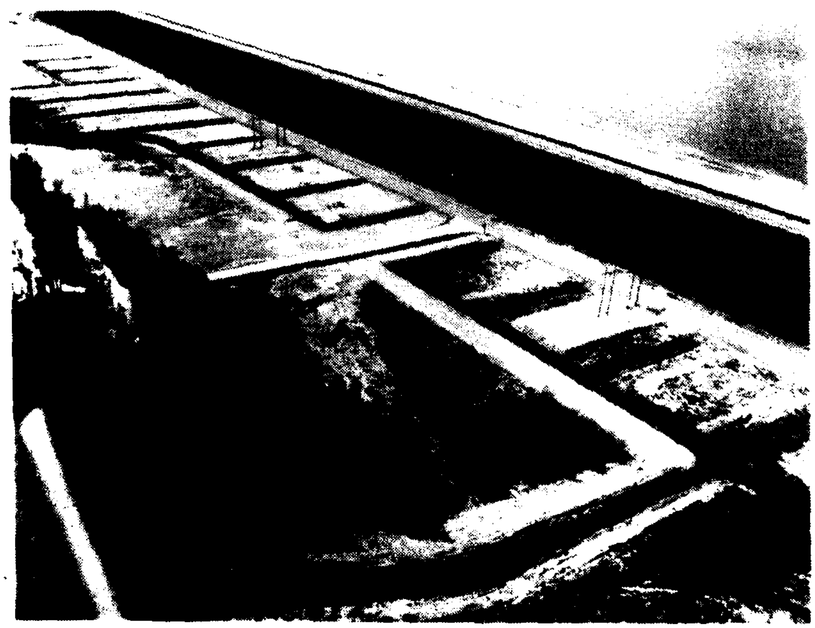


Photo No. 24 - Aerial view showing relief well discharge and collector ditches. Discharge ditch for relief well No. 19 is pictured in extreme right of photo. 8 May 1957.

Photo No. 25 - Wave erosion scarp along embankment upstream chalk berm. Oct 1966.

Photo No. 26 - Reservoir ice thrust against riprap along embankment upstream berm. Mar 1976.

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